HIGHWAY RESEARCH RECORD

Number 120

Aggregate Characteristics and Examination

3 Reports

Presented at the
44th ANNUAL MEETING
January 11-15, 1965

SUBJECT CLASSIFICATION
32 Cement and Concrete
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of the
Division of Engineering and Industrial Research
National Academy of Sciences—National Research Council
Washington, D. C.
1966

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Foreword

It is estimated that as much as 30 cents, or even more, of every highway construction dollar is spent on mineral aggregates. The materials engineer is thus faced with a grave responsibility to insure that this very substantial portion of the total investment is wisely spent. Selection of mineral aggregates suitable for the intended use presents a continuous challenge. His background of experience with aggregates sources of proven performance may be of small avail when he is faced with the necessity of evaluating untried materials. This necessity is becoming more commonplace as present sources are depleted, leading him to seek quick and trustworthy procedures for predicting service performance of new aggregates. Work reported here is part of a continuing effort to provide solutions to the many problems faced in the selection of mineral aggregates.

The first report deals with development of a procedure to determine the freeze-thaw behavior of concrete aggregates, using a technique much abbreviated as to equipment needs and time for testing when compared with practices currently employed by many agencies. Concretes containing a variety of aggregates were evaluated using conventional freeze-thaw techniques and the results compared with the proposed simpler procedure employing an ordinary deep-freeze unit and Whittemore strain gage. The procedure is still under development and gives promise of success by determining either the length change vs temperature or length change vs time curve during a single freeze. Discussion is offered cautioning that it may be necessary to isolate the contribution of the portion of the curve attributable to normal coefficient of thermal expansion. The full text of this paper is now available as an Interim Report in National Cooperative Highway Research Report No. 12.

The second report surveys the often overlooked contribution that the petrographer can provide as a complement to the standard physical acceptance tests of concrete aggregates and is written to be valuable to the general interest reader who desires background information, as well as to the specialist. A petrographer, particularly one having a background of experience in concrete technology, is often able to use his specialized techniques of examination to provide valuable explanation of apparent anomalies between predicted and actual service behavior. Questions regarding "problem" aggregates may well be resolved by the competent petrographer using either cursory methods, examination with the petrographic microscope, or more advanced techniques. The paper gives details on the copper nitrate staining test used for examination of carbonate rocks.

Present day high-speed rubber-tired traffic contributes to development of dangerously slippery pavement surfaces at some locations. These slippery conditions have often been traced to use of carbonate aggregates, limestone or dolomite, in the surface course. The third paper reports extensive studies showing that pavement surfaces containing certain types of carbonate aggregates are not susceptible to polishing. Such aggregates possessing appreciable amounts of insoluble residue of sand size (0.05 to 2.0 mm) display satisfactory skid-resistance. The NCSA Slipperiness Testing Apparatus used in the research is described in detail.

Contents

Identification of Coarse Aggregates That Undergo Destructive Volume Changes When Frozen in Concrete

RICHARD D. WALKER
Associate Professor of Civil Engineering, Virginia Polytechnic Institute, Blacksburg, Virginia

ABRIDGMENT *

•THE OBJECTIVE of this project was to develop a quick test method to distinguish deleterious particles in aggregate and predict their behavior in concrete subjected to freezing and thawing.

Thirty-two different concretes were used to fabricate a total of 281 3- by 3- by 16-in. specimens. Eight coarse aggregates were used ranging from trap rock and limestone crushed stones to a variety of gravels from glacial and nonglacial sources. Concretes were made from the whole coarse aggregates as well as from certain of their individual constituents.

Most of the concrete specimens were exposed to alternate cycles of freezing and thawing while in water in accordance with ASTM test designation C 290-61T. The specimens were measured for length, weight, and dynamic modulus at the end of specified numbers of cycles. Also, length change measurements were made with a Whittemore strain gage at approximately 2-deg intervals during most of the initial freezing-and-thawing cycle as well as during several other cycles. The freezing rate was approximately 20 F/hr.

Companion specimens, in some cases, were also exposed to a single slow freeze cycle in a deep freeze unit. Length change and temperature measurements were made at 10- to 15- min intervals over a 4-hr period. The freezing rate of this procedure was between 5 and 10 F/hr.

From the data obtained, correlations were made between concrete durability as measured by dynamic modulus and certain characteristics of the temperature vs length change curve.

AGGREGATES STUDIED

The eight coarse aggregates used are briefly described in the following.

A. A high quality quartzite gravel from the eastern Piedmont region. This is a hard durable aggregate with an excellent field performance record. It was used both for comparative purposes and for blending with other aggregates.

B. A high quality trap rock crushed stone from the eastern United States.

C. A crushed limestone having unknown field performance from southeastern Canada.

D. A gravel of very heterogeneous composition from a river terrace of the glaciated midwest. This aggregate has a fair field performance record.

E. A blast furnace slag from the eastern United States.

F. A midwestern river gravel, composed mostly of chert, having a poor field performance record.

G. A chert gravel from the Ohio River having an extremely poor field performance record.

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*This is an abridgment of a paper presented at the 44th Annual Meeting. The complete paper as then presented is available from the HRB at cost of reproduction and handling. However, an interim report of this project (NCHRP 4-3(1) FY '63) has been published as NCHRP Report 12.

Paper sponsored by Committee on Mineral Aggregates.

H. A reject midwestern glacial gravel which floated in a commercial heavy media plant operating at a media gravity of 2.55. This aggregate has no field performance record because it is not used in pavement structures.

In addition to these eight aggregates, individual rock types were separated by hand from aggregates D and H in sufficient quantity to be used in concrete. Aggregates D and H were the most heterogeneous gravels obtained and the only ones from which sufficient quantities of many different rock types could be obtained. The other six aggregate sources were of more or less uniform composition. The individual rock types separated were as follows: (1) high calcium limestone, (2) chert, (3) limey chert, (4) calcitic siltstone and sandstone, (5) quartzite, (6) non-end members of limestone and dolomite, (7) dolomite, (8) weathered rounded trap, (9) shale, and (10) granite.

Concrete Mix Design

All concrete mixes were designed using a blend of three brands of low alkali type I cement, a cement factor of 5.5 sk/cu yd, and air content of 5.5 percent, and sufficient water to obtain a 3-in. slump. A single high-quality quartzite sand was used throughout the entire project. All course aggregates were vacuum saturated and all specimens were cured in lime water for 13 days.

Results and Discussion

Two terms must be defined before the results of the study can be presented and discussed:

1. The minimum 5 F temperature slope, b_1, is the minimum slope that can be found, within a 5 F or more range, on the length change vs temperature curve obtained during the first freeze of each specimen. It is in units of 10^{-4} in./deg F measured over a 10-in. gage length.
2. The time slope, b_t, is the minimum slope that can be found within a 20 min (⅓hr) or greater range on the length change vs time curve for the slow freeze cycle specimens only. The units of b_t are 10^{-4} in./hr (measured over a 10-in. gage length).

Table 1, herein (Table 3 of original paper; Table 5, NCHRP Report 12), gives the relationship between DF_{100} and b_1 for all specimens that underwent the alternate freezing and thawing cycle procedure. The values are the averages of all specimens for the particular mix design. Although a certain amount of variability is evident, a fairly good separation is obtained between groups I and II and groups II and III.

Figure 1, herein (Fig. 3 of original paper; Fig. 6, NCHRP Report 12), shows the relationship between DF_{100} and b_1 for the concretes made with the individual particle types. With the exception of aggregate U, a shale, a fairly good relationship is shown.

Figure 2, herein (Fig. 8 original paper; Fig. 11, NCHRP Report 12), shows the relationship between DF_{100} and b_t for the specimens frozen in the deep freeze. Omitting points F-2 and L for purposes of illustration, an extremely high correlation coefficient is obtained. If b_t were a good prediction of the quality or durability of concrete exposed to freezing and thawing, specimens could be tested using only a deep freeze unit, a Whittemore strain gage, and an instrument for keeping time. Expensive temperature measuring equipment would thus not be required.

On the basis of these results, it is felt that one or both of the test procedures described has excellent potential as at least a preliminary test to separate potentially bad aggregates from good aggregates. In no case did a mix design have a b_1 or a b_t value of less than zero, indicating that dilation never took place, with aggregates falling in the good category. Perhaps if a b_1 or b_t value of less than zero were obtained, it might be decided that more lengthy tests might be performed. All of the traditional freezing-and-thawing tests and the one cycle freeze test described by Powers (1) are time-consuming. It is hoped that future testing will show that a test of the nature described in this study will prove practical.

REFERENCE

1. Powers, T. C. Basic Considerations Pertaining to Freezing-Thawing Tests. Proc. ASTM, Vol. 55, pp.1132-1155, 1955.

TABLE 1

RELATIONSHIP BETWEEN MINIMUM 5 DEG TEMPERATURE SLOPE, b_1 (10^{-4} IN./DEG F) AND 100 CYCLE DURABILITY FACTOR, ALL MIX DESIGNS

Group I Extremely Bad[a]			Group II Bad[b]			Group III Fair[c]			Group IV Good[d]		
Mix	DF_{100}	b_1[e]	Mix	DF_{100}	b_1[f]	Mix	DF_{100}	b_1[g]	Mix	DF_{100}	b_1[h]
F-1	3	-2.602	F-2	12	+0.075	D	66	+0.102	A	97	+0.882
F-3	3	-0.549	I	25	-0.398	E	59	+0.143	B	98	+0.835
G	1	-5.768	J-1	9	-0.014	J-3	44	+0.489	C	84	+0.316
H	5	-2.072	J-2	14	+0.063	J-6	66	+0.450	K-1	85	+0.029
N	4	-1.316	J-4	12	-0.073	R	51	+0.178	K-2	93	+0.010
W	1	-3.274	J-5	7	-0.220	S	46	+0.101	K-3	94	+0.167
			L	10	-0.300	U	40	+0.642	M	95	+0.291
			O	8	-0.863				Q	95	+0.264
			P	27	-0.332				T	98	+0.698
									V	95	+0.813
Avg.	3	-2.596	Avg.	14	-0.229	Avg.	53	+0.330	Avg.	93	+0.431

[a] DF_{100} = 0-5.

[b] DF_{100} = 6-30.

[c] DF_{100} = 31-80.

[d] DF_{100} = 81-100.

[e] Ninety percent confidence limits: -4.095 to -1.097.

[f] Ninety percent confidence limits: -0.409 to -0.049.

[g] Ninety percent confidence limits: +0.139 to +0.461.

[h] Ninety percent confidence limits: +0.233 to +0.629.

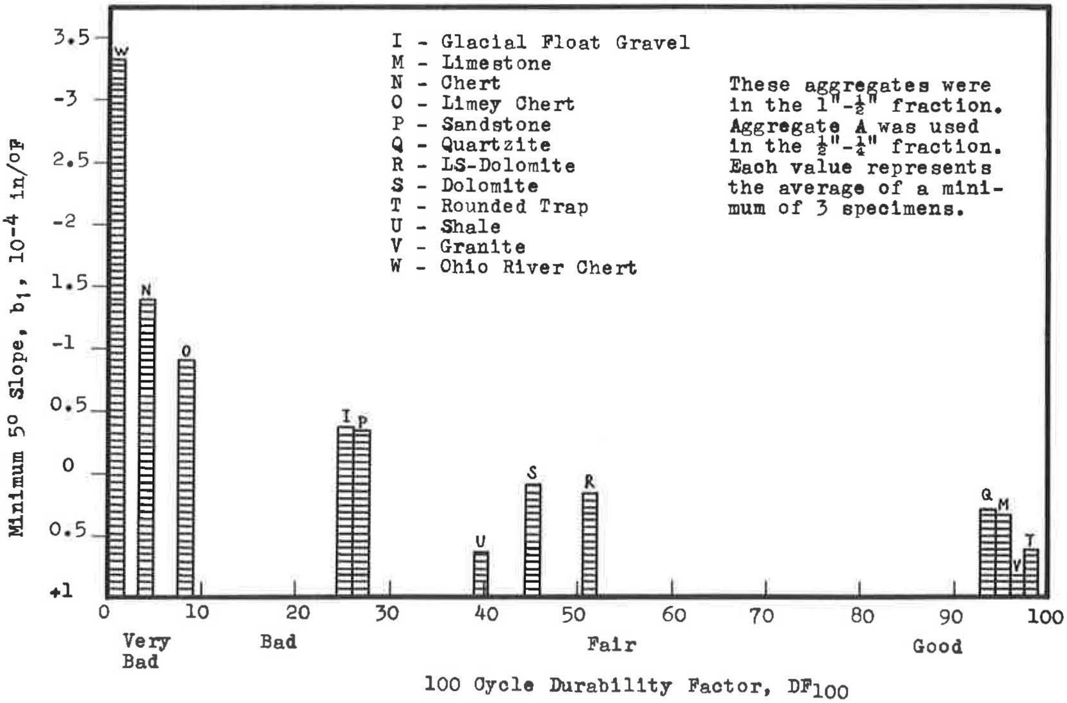

Figure 1. Relationship between DF_{100} and b_1 for individual particle types.

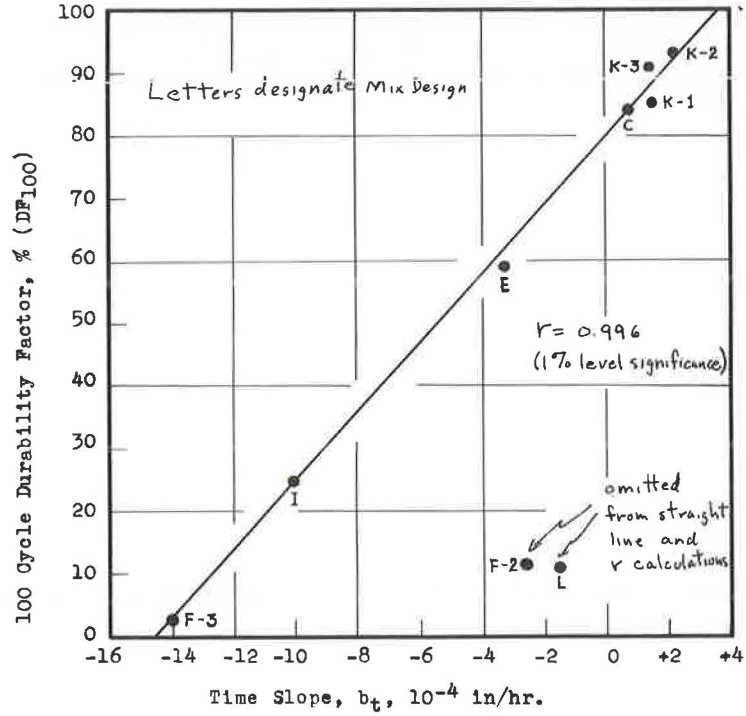

Figure 2. Relationship between DF_{100} and b_t for specimens frozen in deep-freeze.

Discussion

BAILEY TREMPER, Consulting Engineer, Riverside, California—Professor Walker has adopted the term b_1 as a measure of length changes in concrete during the first freeze of each specimen. A negative value of b_1 signifies an increase in length. Conversely, if b_1 is positive a decrease in length is indicated.

The value of b_1 as used by Walker is evidently the sum of changes due to thermal effect and that produced by dilation of the specimen due to the effect of freezing. Since the thermal coefficient indicates shortening during descending temperature, its value is positive. The thermal value should be substracted from the total observed length change to obtain the value of dilation due to the freezing effect alone.

The thermal coefficient of expansion of concrete is generally in the range of 3 to 6 × 10^{-6} which is equivalent to 0.3 to 0.6 × 10^{-4} measured over a 10-in. gage length, which is the unit used by Walker. These are the values observed after moisture in the gel pores and the capillary pores has reached equilibrium following a change in temperature. Helmuth (2) has shown that when hardened cement paste has been subjected to a change in temperature at a rate comparable to that used by Walker, the observed thermal coefficient is approximately double the value found after moisture equilibrium has been established.

An increase in the freezing rate of concrete would be expected to increase the apparent thermal coefficient, and presumably the amount of dilation also, but the relative effect on the two factors would not necessarily be the same. Examples of the effect of freezing rate on b_1 for two concretes are shown in Figures 6 and 7 of original paper (Figs. 9 and 10, NCHRP Report 12). For concrete F_2, b_1 was zero under rapid freezing but increased to +0.19 under slow freezing. For concrete L, b_1 was -0.3 under rapid freezing but increased to +0.2 under slow freezing. Although these results may indicate that slow freezing was less destructive, such a conclusion cannot be confirmed without knowledge of the effect of freezing rate on the apparent thermal coefficient.

Professor Walker did not report the thermal coefficients of the concretes he tested, but since the values of b_1 as indicated in Table 1, herein, were obtained under decreasing temperature rates of from 13 to 26 deg F/hr, the change due to the thermal effect alone could well have been equal to the observed values of b_1 as reported for group IV (good) aggregates. In other words, these concretes may have exhibited nothing but the natural thermal change in length, and therefore would be rated as "good" on the basis that they developed no dilation.

For the poorer concretes of Table 1, herein, (those exhibiting negative values of b_1), the value of dilation would be the value of b_1 minus the value of the thermal coefficient and would be considerably more negative than b_1 (would exhibit greater dilation).

Professor Walker states that specimens falling in the fair to good categories did not evidence overall expansion usually. Therefore, he concluded that b_1 would be a better measure of performance. The only aggregate he tested that was known to have a "fair" performance record was aggregate D. (The remaining aggregates listed in the fair category in Table 1 were so placed solely because of results obtained by ASTM Designation: C 290-61T, Rapid Freezing and Thawing in Water.) Concrete D developed a b_1 value of 0.102×10^{-4} in. per deg F. If any reasonable value of thermal coefficient is subtracted from this value the result would be negative, indicating actual dilation, the value of which it should have been possible to measure. Therefore, Professor Walker's choice of b_1 as a measure of performance does not appear to be superior to that of dilation.

If all of the concretes tested by Professor Walker had the same thermal coefficient and if this value were subtracted from the reported values of b_1, the separation into four groups indicating good, fair, bad and extremely bad in Table 1 would not be changed. But, it is reasonably certain that all concretes did not have the same thermal coefficient. Therefore, a grouping according to dilation would undoubtedly have brought about changes in rating. The ratings during the first freeze then would not necessarily agree with that shown by the 100-cycle durability factor.

This brings up the question of the validity of using the test for rapid freezing and thawing in water as a measure of field performance. Professor Walker indicates that he questions its validity. Powers (1) has questioned the rapid freezing methods because of the high rate of temperature drop and the unrealistically high internal hydraulic pressures that are developed. Other features of the test as conducted which are not in accordance with the principles set forth by Powers are

1. The use of 1-in. maximum size aggregate particles unless this is the largest size that will be used in the work.
2. Testing the concrete in a saturated condition, whereas in the work, most concrete is afforded some opportunity to dry between the time of construction and exposure to freezing conditions.
3. The fact that the test gives no indication of the performance of partially dried concrete after it is exposed to moisture during winter conditions which vary from location to location.

Professor Walker has stated that his objective was to develop a quick, simple and inexpensive method of test to distinguish deleterious particles in aggregate and predict their behavior in concrete subjected to freezing and thawing. It is not difficult to devise such a test that will distinguish between aggregates of unquestionably poor performance and those of unquestionably excellent performance. The problem lies in rating marginal aggregates that may give satisfactory performance under moderate exposure but not under more severe freezing-and-thawing conditions. It is not believed that Professor Walker's approach is promising for the latter situation, because of the considerations outlined previously.

Although low testing expense certainly is to be desired, the fact that substantial additional costs may be involved in transporting large quantities of aggregates tends to overshadow possible savings in testing expense. Usually, automatic recording methods for measuring length changes will reduce the operator's time sufficiently to make such methods less costly than manual measurements.

Finally, it appears that true economy will result only if the length and severity of winter exposure at a particular construction site is evaluated to select the appropriate test procedure for such conditions.

Reference

2. Helmuth, R. A. Dimensional Changes of Hardened Portland Cement Pastes Caused by Temperature Changes. Proc. Highway Research Board, Vol. 40, pp. 315-356, 1961.

R. D. WALKER, Closure—The author greatly appreciates the thorough review given to this paper by Mr. Tremper. In the light of his discussion it is felt that certain points should be emphasized.

The object of this study was to develop a simple, short, and inexpensive test procedure for evaluating aggregates. The data shown in Figure 2, herein, indicate excellent promise of correlation between time slope, b_t, and durability factor. With this procedure, only time and length change are being measured; thus, a correction for thermal coefficient of the concrete would not be practical. With each concrete made of the same constituents except for the coarse aggregate, the correction would be minimal.

Mr. Tremper points out that automatic freeze-thaw test specimens showed lower temperature slope (b_1) values than those frozen in the deep freeze unit. This is certainly due to a lower freezing rate and to the specimen being frozen in air rather than in water. However, placing the specimen in a 0 F environment should provide reasonably standard freezing conditions which are essential in the development of a test such as this.

Concerning severity of test, as long as correlation of results is ultimately based on field conditions, it should not matter, in most cases, how severe the test is if it will permit differentiation of performance. An aggregate such as the slag (aggregate E) used in this study would be an exception, due to its large pore sizes. It had a durability of only 59, yet in Maryland it has an excellent field performance record.

Mr. Tremper commented on the inadvisability of restricting the maximum aggregate size to 1 in. in the tests described. The test could easily be modified to use larger specimens which could accommodate larger aggregate; however, our studies indicate that there is at least some doubt as to the effect of top size of aggregate on results. As for testing the concrete in a saturated condition, the pros and cons could be presented in a lengthy discussion. In this study saturation was used to promote greater standardization of the test as well as to permit quick results.

This study does not provide a solution to all our problems but only a beginning towards a standardized, simple test that will be of value for at least the preliminary separation of the very bad and very good aggregates. It is believed it can accomplish more than that. A standard test cannot hope to duplicate field environment exactly. A test which does attempt to do this, in the author's opinion, will lose much of its practical value.

Petrographic Examination of Natural Concrete Aggregates

L. DOLAR-MANTUANI
Structural Research Department, Hydro-Electric Power Commission of Ontario, Toronto, Canada

Methods used in the petrographic examination of coarse and fine natural concrete aggregate are described. Features of aggregates, based on their composition, structure, and texture, influencing the quality of a concrete are discussed. In addition to standard petrographic methods the copper nitrate staining test has been found useful for both identification and appraisal of carbonate materials. Attention is also drawn to the need for further testing of special problem aggregates, such as the alkali-silica-reactive or the alkali-carbonate rock-reactive aggregates and those containing clay minerals.

The role of the petrographer is stressed in complementing and interpreting standard acceptance tests designed to determine the suitability of a given material as concrete aggregate.

•AGGREGATES for large concrete structures must be carefully selected in order to minimize service deterioration due to possible aggregate inadequacies. When natural aggregates are used it is sometimes necessary to consider a large number of gravel and sand deposits and rock quarries. Therefore, for quick yet satisfactory indications of the best deposits for intensive study, simple short-term standard acceptance tests which include cursory petrographic examinations are used.

This paper describes petrographic examinations of rocks and minerals that have been applied for many years in the preliminary analysis of aggregates intended for use in both large and small concrete structures by Ontario Hydro (1). The methods outlined reflect the fact that the aggregates examined have been mainly from the Canadian Shield, and from a region of Paleozoic and Cenozoic sediments.

Petrographic laboratories dealing with a considerably greater number of samples than Ontario Hydro, must usually simplify these methods even further. Examples of routine methods are those used by the Ontario Department of Highways and described by Hewitt (6). The methods outlined in this paper may, however, be included in every petrographic examination of concrete aggregate, and may also serve for examination of rocks used in other structural fields where special requirements demand additional examinations or emphasis of other properties.

Basis for Selection of Methods

Although intended mainly to classify and describe materials in their natural state, routine petrographic examinations usually include consideration of their behavior as structural materials. This point is emphasized in the following quotation from ASTM Designation C295-54, Recommended Practice for Petrographic Examination of Aggregates for Concrete (20):

> Identification of the constituents of a sample is usually a necessary step toward the recognition of the properties that may be expected to influence the behavior of the material in its intended

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> use, but identification is not an end in itself. The value of any petrographic examination will depend to a large extent on the representativeness of the samples examined, the completeness and accuracy of the information provided to the petrographer concerning the source and proposed use of the material, and the petrographer's ability to correlate these data with the findings of the examination.

Furthermore, as stated by F. S. Fulton (3): "A knowledge of the ways in which rocks are formed, and of the various natural processes whereby their original characteristics are altered may lead to a better understanding of those intrinsic properties which determine the suitability of a rock also as a source of concrete aggregate."

Requirements for aggregate vary depending on the end use of the concrete. Aggregate characteristics determined petrographically include structure, texture and composition of the aggregate constituents. These may affect desirable properties in the concrete such as compressive and flexural strength, abrasion resistance, toughness, and durability. Durability implies resistance to climatic conditions, such as temperature changes, wetting and drying, and freezing and thawing. The concrete may also need to be especially resistant to heat, or to actions from such sources as polluted air, sulfate waters, organic acids, and alkalies. Concrete aggregates must be both physically and chemically compatible with the other constituents of the concrete.

The properties of aggregates may be either inherent in rocks and minerals or have been acquired through processes of alteration or weathering during geologic time. Material of acceptable quality in its fresh state may in the weathered state be completely unsuitable. The effect of weathering, although very important, is unfortunately difficult to describe and appraise, because the changes in structure, texture, and composition are continuous.

Economy may not be of immediate concern to a petrographer, but it can be the deciding factor in source selection from among two or more materials of equal or almost equal quality.

Numerous physical and some chemical standard acceptance tests are applied in appraising a material for structural purposes, with petrographic examination but one of the required steps. There are two main differences between the two approaches. First, unlike standard acceptance tests, petrographic examinations are not performed in a strictly prescribed manner. Second, petrographic examinations give information on the composition and quality of individual particles, whereas most standard acceptance tests provide data on bulk samples only.

The effectiveness of various petrographic methods depends, of course, on the materials to be studied. Established techniques for sedimentary rocks (Krumbein and Pettijohn, 7) are the most applicable not only because much concrete aggregate is of this origin but also because mortar and concrete products may be considered artificial forms of sandstones and conglomerate or breccias, respectively.

In carrying out petrographic examinations it may be necessary to focus attention on a particular characteristic of a material either because it is needed or because it may be harmful and is difficult to detect. Examples of this would include porosity of vuggy dolomites or of carbonaceous shaly sandstones, or special types of opal or chert in carbonate rocks.

More detailed examinations involving little more time may frequently yield important results. In some instances the petrographic examination will indicate the need for extensive physical and chemical testing to assess fully some potentially deleterious characteristics (10). The size and importance of the proposed structure also greatly influences identification for concrete aggregates.

Whenever possible, materials for structures are compared with materials of known satisfactory records in similar structures. This comparison may have to be based entirely on petrographic methods because no other method can provide such exact identification for concrete aggregates.

Knowledge that proposed and known materials are from the same source is alone usually insufficient to establish the quality of the proposed material, because frequently variations occur both vertically and horizontally throughout deposits. This applies particularly in gravel and sand deposits. In quarries, composition and quality are usually more uniform. Even if materials in a structure have performed satisfactorily, the older the structure the less likely that present materials from the same general source will correspond exactly to those in that structure. Furthermore, as appropriately pointed out by ACI Committee 621(21), modern concreting techniques may be a factor in a material's suitability, "...it should not be overlooked that unfavorable service of old concrete without entrained air may not accurately indicate performance of concrete with the benefits of proper air entrainment."

For the foregoing reasons, decisions on aggregate materials are based more often on comparisons of economically available materials with other known materials, than on appraisals according to a given standard. Generalized standards cannot reflect the different criteria that should be applied, as these criteria are dependent on the end use of the material. In addition, the reasons for acceptance or rejection of a material or a source for one application may not be valid for other applications.

Petrographic tests often help to explain failures that occur in other tests by revealing the materials which cause failure. From experience gained through examination of materials that have failed in standard acceptance tests, the petrographer becomes more adept at appraising aggregates for structural purposes. The identification of harmful materials in a deposit may lead to recommendations of beneficiation procedures which would make acceptable an originally unsuitable aggregate (8).

Often in the formulation of specifications, petrographic findings make it possible to set limits on permissible quantities of harmful constituents of aggregates to be used for particular structures. The limits are established by using data from previous studies and information on the performance of existing structures. This requires close cooperation between the petrographer and the concrete technologist. The establishment of these compositional limits can be of great value because conformance with them may obviate the need for lengthy physical tests on aggregates.

In general specifications, reference is usually made to certain deleterious aggregate constituents such as coal, lignite, shale, and unsound chert, and the percentage limits tolerated are stated. Checking for conformance with these specifications inevitably involves petrographic examinations, but because the terms used in specifications are too vague for precise identification of the materials, judgment also must be used.

SAMPLING

In large agencies with extensive job specialization, trained field personnel normally do the sampling (14). In special instances involving the geological position of a sample to be tested, or collection of a particular material, a petrographer does the sampling. Procurement and retention of photographs of the exact areas sampled are advisable, especially where working pits or quarries are involved. Visual comparisons by means of such records, of the sampled area with those that may later be developed, may obviate the need for testing samples from the aggregate supply area.

According to long-established practice and in conformance with the specifications used in North America, the No. 4 sieve (4.76-mm size) is used for the first main separation into coarse and fine aggregates. Normally, the samples are separated into fractions by means of standard sieves so that the particles of the different constituents present may be counted rather than weighed.

The size of sample required depends on the maximum particle size of the material available to the petrographer. Often, the coarsest fraction petrographically examined is that specified for the structure. More frequently, however, the coarse aggregates examined are limited to three fractions consisting of gravel or crushed rock in the sizes for standard laboratory acceptance tests—fractions retained on $\frac{3}{4}$-in., $\frac{3}{8}$-in., and No. 4 sieves. Usually, approximately 50 lb coarse aggregate and 2.5 lb fine aggregate quartered down from the entire representative sample are sufficient to provide 200 particles of each fraction for petrographic examination. In this way, enough material is available to insure that even the $\frac{3}{4}$-in. fraction of a normal-weight coarse aggregate and the No. 8 size fraction of fine aggregate contain all essential constituents. If the material is not deficient in one or more size fractions, these quantities of both coarse and fine aggregate will usually be sufficient for any special petrographic tests or examinations.

Fine and coarse aggregates are always appraised separately because the size of the particles may influence the choice of not only the physical test methods applied but also of the petrographic techniques.

COARSE AGGREGATE: FEATURES AND TEST METHODS

Coating

Before the petrographic examination is started, the aggregate sample is visually inspected to ascertain whether dust is present in an excessive amount. Rock dust adhering to the coarse aggregate as a form of coating (on wet particles in particular) may be crusher dust that accumulates during processing. It may result from insufficient separation of fine pit material, or it may accumulate from soft fine-grained particles that gradually disintegrate. Such accumulations occur more frequently in gravel from pits that contain silty and clayey seams and layers. Silty and clayey material may adhere to gravel surfaces so firmly that it is not removed by normal washing. Dust in excessive amounts may be concentrated in those parts of an aggregate pile that have been stocked longest.

Dust coatings are usually reported as materials passing the No. 200 sieve, and percentage limits are set in most specifications. If information on the amount of dust, where excessive, is not available, the amount is determined in the petrographic laboratory. In such instances and also if the amounts are borderline, additional information on the composition of the dusty coating can be useful. The dusty coating is collected by washing the sample over appropriate sieves, the finest being the No. 200. The wash-water is collected and evaporated, and the residue weighed. Then on an adequate sample of the minus 200 portion, the hydrochloric acid-insoluble test is performed, followed by the barium chloride test for the presence of sulfates. The acid-insoluble residue, mounted on slides, is examined microscopically, special attention being given to any clay minerals present.

The petrographic study of more or less firmly attached coatings is made on single particles by means of a lens or microscope.

Particle Shape

Usually, particle shape, which is most important for crushed aggregates, is the first feature to be determined. This determination and the counting of the 200 particles of each fraction are done together. Because particle shape of the crushed aggregate can be corrected (5), a shape analysis involving detailed measurements is normally not attempted unless the amount of flat and oblong particles, which are undesirable, approaches the permitted limits. Particle shape is classified under two general characteristics (19): roundness, i. e., the particle is rounded, subrounded, subangular or angular; and sphericity (the three-dimensional shape or shape proper) i. e., the particle is cubic, flat or oblong. For a quick determination of the roundness, comparison of the particle shape with appropriate charts is adequate (16). A smooth surface texture might suggest a greater roundness of particles than actually exists. Sphericity can be determined by the use of a simple divider set at a given ratio. The U. S. Corps of Engineers and Ontario Hydro have adopted the ratio 1:3 both for width to length and for thickness to width to distinguish between cubic, and either flat or oblong particles. For road material the ratio 1:4 (22) or 1:5 (23) is required, and the determination of fractured faces is frequently important in the assessment of the roundness of particles. Crushed particles are defined as those which have at least one or two fractured faces (20).

General Observation

The initial megascopic examination of coarse aggregate involves recognition of each essential constituent and determination of its textural and structural features, and also of those accessory constituents the properties of which might determine the quality of the aggregate as a structural material. A magnifying lens, a hammer, a knife, water, and dilute hydrochloric acid are the simplest aids used. The $^3/_8$-in. fraction is found most convenient for first study because this size permits relatively easy identification of the rock constituents as well as of the different materials which may form part of the particle surface.

Although classification of aggregates by composition is a straightforward petrographic procedure, the details required for the different varieties may vary considerably from sample to sample. For aggregates composed mainly of rock of one type such as that obtained from one quarry or an indurated rock excavated from a limited area, more detailed classification of the various features of that particular rock are necessary. The need for details applies also for gravel derived from a pit very uniform in composition.

Surface Texture

Although surface texture no doubt influences the bond of both coarse and fine aggregate particles to cement paste, and accordingly can adversely affect the strength of concrete, standardized techniques for examination of this feature have not yet been developed (15). The description of aggregate-surface textures, based on visual observation, is usually limited to general terms such as granular, crystalline, glassy, polished, smooth, rough, dull, or surface marked. The surface may be described as furrowed and grooved, scratched, ridged, or dented; striated, etched, frosted (minute irregularities), or pitted (larger scattered irregularities) (7).

Visual Estimation of Other Surface Features

In the megascopic examination of aggregates, special attention is given to various objectionable impurities in the rocks or minerals, such as shaly seams, rusty accumulations, inclusions of chert, sulfates and sulfides, and more or less firmly adherent coatings. To evaluate the degree of harmfulness, the amounts of the various impurities are often estimated as percentages of the particle surface, the surface layer being assumed the most important feature in the eventual reaction with the cement paste. Such estimates can be made quickly and with reasonable accuracy, provided that a color difference exists. For proficiency in estimating, experience is essential, and can be acquired by comparison of estimated results with results obtained by the use of adequate visual aids (25), or by comparison of estimated percentages of rock constituents or other parameters in thin sections with those obtained microscopically by the point-count method.

Quality

Although in petrographic studies of an aggregate the influences of the physical and chemical properties of a constituent are usually appraised separately, the presence of a particular constituent may be both physically and chemically undesirable. Furthermore, the same constituent may be harmless under certain conditions but harmful under others, or even beneficial for one purpose but detrimental for another.

Harmful physical properties, evidenced as brittleness, lamination, and friability of constituents, are generally more readily established than harmful chemical properties. Physical properties depend to a great extent on the original structure and texture of the particle, and on the abundance and nature of pores and internal fractures in the material. Special attention must therefore be given to such features as porosity, vesicularity, schistosity, presence of micaceous minerals, poor bond between grains and between grain assemblages after weathering, and insufficient cementation. For classification of the physical quality of individual particles the terms used are good or satisfactory, fair, and poor (12).

To appraise physical quality, in the course of routine examinations simple tests are performed on individual particles, for example, light hammer tapping or, for finer sizes, breaking of particles with the fingers. To determine whether particles are soft and/or friable, a varying pressure is applied to break pieces with a hammer, using a rotatory motion. The force exerted depends on the particle size. Rapid absorption of a water drop on a rock surface may be the first indication that a particle is physically and perhaps also chemically objectionable.

Although subjective, such physical testing substantiates visual findings and has the advantage of involving entire individual particles. It helps to detect the presence of clay or mud balls or other similar incoherent particles and may indicate the degree of weathering of particular constituents of a sample. It may show the need for other specific standard tests to appraise the aggregate's physical quality.

The scratch hardness method (ASTM Test C 235-57, 20) in which individual particles are scratched on several faces with a brass rod that resembles a sharpened pencil may also be applied. The particles remain practically undamaged, thereby permitting the same samples to be tested by other methods. The scratch test with a knife is an important method for quick distinctions between chert and carbonate rocks of the same color, and between dolomite and chert containing carbonate inclusions.

In the classification of an aggregate by chemical quality, the observer must concentrate on unsound materials that are part of the original rock composition or have been produced by various alteration and weathering processes in later stages of the rock's history. Such substances are limonite, clay minerals, chert, certain zeolites (laumontite, heulandite) and volcanic glass. According to the literature, the number of minerals and rocks that contain harmful substances seems to increase from year to year, partially because more nearly exact identification of potentially harmful substances in structural materials is now possible. No doubt some findings on the general quality of rocks and minerals must continuously be revised as new information becomes available. Aggregate constituents are classified chemically as being either innocuous or deleterious, or as suspected of being deleterious if their effects cannot be established exactly.

From the numerical distributions of the various particles in each fraction, tables of aggregate constituents with varieties subdivided according to their expected qualities are prepared. The weighted average for the whole sample is calculated by taking into account the weighted grading.

Staining Method

For speeding up the megascopic identification of the mineral constituents and providing additional information on the quality of an aggregate, staining techniques are very useful. In the structural branch of applied petrography they are mainly conducted on carbonate rocks and minerals. In the staining method used in the Ontario Hydro Laboratory (2), 200 particles of each fraction are immersed at room temperature in a copper nitrate solution[1]. Consequently most fractions of a sample are examined in two sets of 200 pieces each, one set by the staining method and the other by routine examinations, each revealing different properties of the same constituents. Therefore, 200 particles for each fraction is considered sufficient for routine examinations, even though the ASTM prescribes petrographic examination of at least 300 particles. (C 295, 20).

The prolonged copper nitrate staining test shows not only the differences between the carbonate and noncarbonate rocks, and between carbonate rock types, the limestones and dolomites, but also in the carbonate rocks it shows the presence, type and development of silicate impurities such as quartz, feldspar grains, chert, and clayey films, seams and accumulations, each of which has a bearing on the quality of aggregates. Not only can these impurities be identified by absence of stain but also shaly accumulations and shaly particles may disintegrate to various degrees. Scaling, splitting, laminating and irregular cracking indicate unsatisfactory quality (Figs. 1 and 2). Examination of the stained gravel fractions before petrographic examination of the unstained sample has proved advantageous because some properties and varieties of the materials are more easily recognized after staining.

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[1] Fifty-one grams hydrated copper nitrate, $Cu(NO_3)_2 \cdot 3H_2O$, are dissolved in 200 ml of water. Before washing the specimens in tap water at the end of the test, they are briefly immersed in a strong ammonia solution to stabilize the blue stain produced by the complex copper salts.

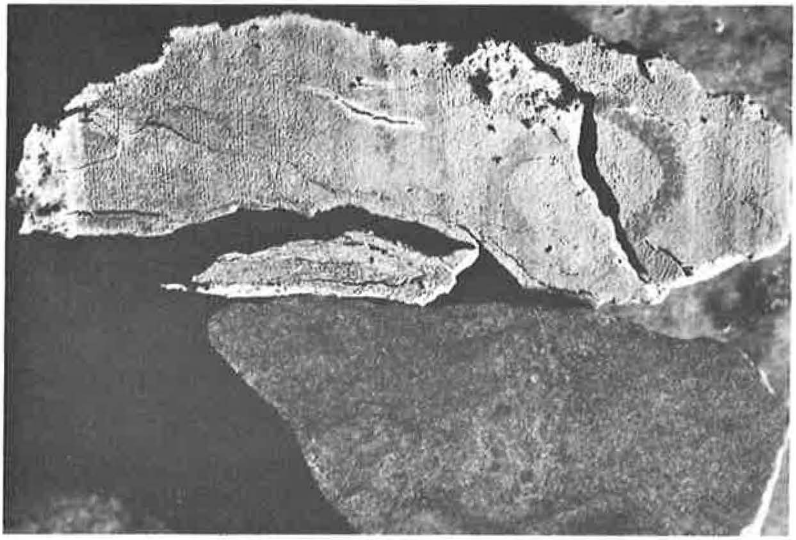


Figure 1. Two pieces of shaly (argillaceous) limestone obtained by cutting through a single particle; lower piece (not stained) with darker shaly, hardly recognizable stripes; upper piece with numerous splits and cracks caused by 16-hr copper nitrate staining test (2). (3.9x)

If a particularly important carbonate material is to be studied or if a carbonate rock is of suspect quality, groups of particles apparently of the same rock variety are stained, or single ¾-in. particles are examined in detail. For the examination of single ¾-in. pieces, a thin section of each particle is prepared, and the staining and the hydrochloric acid-insoluble tests are applied to the remainder of each particle. The acid-insoluble residue is examined microscopically in mounted sections immersed in oils of various refractive indexes and the percentages of the noncarbonate constituents of the rock are estimated.

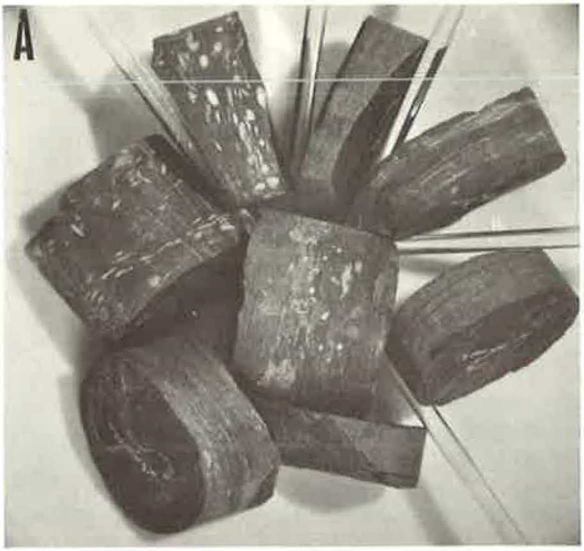
FINE AGGREGATES: FEATURES AND TEST METHODS

The examination procedures for fine (sand-sized) aggregates are similar to those for coarse aggregates except that handling and observation are more complicated because of smaller particle size. It is understandable that many methods have been developed for dealing with sands, particularly if the specific requirements of the various fields such as stratigraphy, glaciology, and economic geology are considered. For examinations of fine aggregates, however, these methods are normally not entirely applicable, and must be modified to meet the needs of the construction industry.

The fractions into which fine aggregates are commonly separated are those retained on sieves Nos. 8, 16, 30, 50 and 100, and passing No. 100 sieve; the last fraction might be divided into two fractions: that retained on, and that passing No. 200 sieve. Although some of these fractions are usually more abundant than others, examination of only one or two of the more abundant fractions does not suffice. The composition of a sand varies from fraction to fraction, and therefore the results obtained on one or two fractions cannot be extrapolated to include the entire sample. In examinations of sands, especially of the coarser-sized fractions even if present in relatively small amounts, a quite detailed classification of the constituents, particularly of those with a poor record, is frequently required.

Microscopic Examination

The 200 particles for each of the fractions from No. 8 to No. 50 are examined in reflected light under a stereoscopic microscope. Either a special needle of bakelite or a brass needle insulated with a rubber sleeve is used for handling the fine-sized fractions. The fractions No. 50 to passing Nos. 100 and 200 are examined in mounted



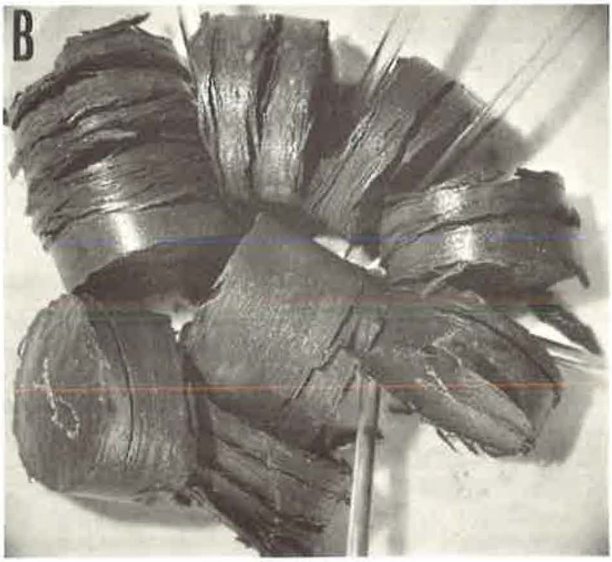


Figure 2. (A) Drill cores of shaly (argillaceous) limestone and calcareous shale. Not recognizable are the amount and development of clayey material in parallel, very narrow seams. White spots are fossil remnants. (B) The same samples, after subjection to an 18-hr copper nitrate staining test. Lamination of shaly seams is clearly visible (2). (0.9x)

sections in transmitted light under a petrographic microscope. Canada balsam is normally the mounting medium but immersion oils may be used instead. Cursory examination of a fraction under the same magnification as used for the next finer size has been found useful in preventing overemphasis of some features which are more visible under the greater magnification.

Additional Tests

In addition to microscopic examinations of original material, the copper nitrate staining test is performed on fractions Nos. 8 to 50, inclusive. The lower limit is the

No. 50 sieve size because finer particles are easily lost in subsequent washing. In order to check the results for the finer fractions, the acid-insoluble test followed by the barium chloride test for sulfates are performed on these fractions at least, and may be warranted for all sand-sized fractions if sufficient carbonate constituents are present.

In more exact determinations, for the larger fractions, those particles not easily identified are examined in thin sections obtained by grinding the tops from 2 to 6 particles attached to a glass slide with Canada balsam. Each particle of carbonate rock suspected to be of poor quality may be dissolved in hydrochloric acid on a spot plate. The effects of reaction of the particles with the acid, and subsequently the remaining insoluble residue, are observed with a microscope. For these examinations stained particles can also be used since the blue color is eliminated by hydrochloric acid.

Quality

Classification by quality of the fine-aggregate constituents may be based largely on the results obtained for the larger-sized particles from the same pit; that is for gravel from gravel-sand pits and for the larger-sized sand fractions or the over-size fractions, if available, from a sand pit. If physical quality appears unsatisfactory, the larger sand fractions are examined by rolling each particle under a spatula. In this way some information is obtained on brittleness, and on the more important characteristic, friability. Distinctly poor particles such as clay lumps and slaking shales are recognizable by their disintegration in water. The coarser fractions are much more convenient than the finer ones for appraisal of the chemical quality of sand when this quality depends on products developed in intensely altered and/or weathered rocks.

Studies of gravel-sand deposits have shown that the disintegration of rocks into sand-sized fractions is frequently beneficial because sand fractions usually consist largely of nonharmful mineral constituents remaining from coarse- or medium-grained weathered rocks. This is particularly true for fine sand fractions. Fine sand fractions may, however, contain undesirable rock constituents such as clay lumps, slaking shales, minutely grained shaly carbonate rocks, chert, and volcanic glass. Some constituents may be harmless in very fine particle sizes but extremely harmful in somewhat larger sizes; for example, opal (9). Certain substances such as micaceous minerals may accumulate in the finest fractions and must be classified as physically poor because their perfect cleavage prevents a good bond to the cement paste in concrete.

Knowledge of the fine-aggregate grading is important because the quality cannot be improved by recombination of the original fractions but only by blending with other sands. The percentage and composition of the minus 200 size fraction are determined, because this fraction can have an appreciable influence on the quality of a concrete.

The results of analyses of fine-aggregate samples are recorded as particle counts of the various constituents, except for the minus 100 and minus 200 fractions for which the quantities for the constituents are only estimated. The results are arranged as described for the coarse aggregate.

Investigations of Manufactured Sand

Because of possible concentrations of harmful material in finer portions during crushing, manufactured sand (and especially crusher fines if used as aggregate) require more detailed examinations. A detailed determination of the composition of crusher fines may be advisable even if the fines are not intended for use as fine aggregate. For comparison of crushed aggregate samples of the same rock formation from different quarries or from different layers of the same quarry, a knowledge of the weighted total composition of crusher fines may provide a better basis than appraisal of the coarse fractions of the crushed rock which cannot be examined in the same detail because of their larger size.

SPECIAL AGGREGATES: DETAILED EXAMINATION

Alkali-Silica-Reactive Aggregates

Special petrographic investigations may be concerned with alkali reactivity, manifested by excessive expansion of concrete. Such expansion can be produced by reaction of chert varieties, silica modifications, acid or intermediate volcanic glass, devitrification products and tuffs of such volcanics, and at least some phyllite (13) with high-alkali cement. Special attention must therefore be given to these substances during any examination of a sample derived from an area suspected of containing such materials.

Alkali-Carbonate Rock-Reactive Aggregate

Particular varieties of carbonate rocks should be added to the list of deleterious aggregates because they cause rapid excessive expansion when used in concrete made with high alkali cement and exposed to humid conditions. These carbonate rocks cannot be identified by standard acceptance tests including those used for establishing excessive expansion, such as the mortar bar test (17). To prevent the use of such highly expansive material in concrete, the development of relatively quick tests that will give reliable results are being studied in various laboratories (4, 18, 24).

Aggregates Containing Clay Minerals

Information on the type of clay minerals present in aggregates such as shales or argillaceous carbonate rocks can be very significant (11). Dependable classification of such clays into the three main groups, the kaolinitic, illitic and montmorillonitic, is usually very time-consuming, however. Fortunately, various tests may be applied to determine if a suspected rock will perform unsatisfactorily because of the amount and type of clay minerals present. In addition to the results of thin-section examinations of suspected rocks and determinations of their acid-insoluble residues in mounted sections, evidence of damage by the prolonged copper nitrate staining and the sulfate soundness tests, and of high absorption, indicate whether the clay minerals will be detrimental.

As further objectionable materials in aggregates are found, new methods for detecting them must be developed and many such methods are started in petrographic laboratories. Frequently these methods become so easily performed and yield such reproducible results that they become standard acceptance tests.

PETROGRAPHIC EXAMINATIONS ASSOCIATED WITH STANDARD ACCEPTANCE TESTS

Often it is desirable to obtain specific information about the composition and fabric of the portion of an aggregate sample that is used for standard acceptance tests. This is feasible, however, for material used for large structures or for special purposes. The task of the petrographer is to examine the sample after, and preferably, also before the acceptance test has been performed to insure that the material used in the various tests is of the same composition as the sample examined petrographically. This step is important because many tests are performed on some fractions only and not on the entire sample.

If acceptance test results are conflicting or unexpected, or indicate that the sample is borderline, additional petrographic information facilitates the decision on acceptance or rejection of a material. Petrographic examinations are mandatory if trends appear that seem related to the location of the sources or to variations in the composition of materials from one source.

In some instances, petrographic examination of a material after acceptance tests have been performed may have little immediate value. Nevertheless, exact identification of any undesirable material may be useful if petrographic examination is later required for quick comparison of seemingly identical materials from the same or nearby sources. The data may provide valuable clues to the types of rocks present in petrologically similar areas. The increasing emphasis now being given to acquiring knowledge of the kind of material that behaves in a particular way, rather than repetition of empirical tests must be considered a forward step.

CONCLUSION

Petrographic examinations complement standard acceptance tests. Such examinations may confirm the results of acceptance tests and indicate not only the additional tests required but also which tests can be omitted. The petrographer greatly accelerates and facilitates the application of information obtained from either good service performance of material that was expected on the basis of laboratory testing, or poor service performance of material that gave good results in physical tests. Petrographic description of a material is not an end in itself, but must be related to the service features required. Constant close cooperation between petrographer and concrete technologist is therefore desirable.

REFERENCES

1. Dolar-Mantuani, L. Petrography Aids Study of Concrete Aggregates. Ontario Hydro Res. News, Vol. 5, pp. 22-27, 1953.
2. Dolar-Mantuani, L. Concrete Aggregate Examination by Prolonged Copper Nitrate Staining Method. Ontario Hydro Res. News, Vol. 14, pp. 2-14, 1962.
3. Fulton, F.S. Concrete Technology, A South African Handbook. The Portland Cement Inst., Johannesburg, South Africa, 1961.
4. Hadley, D.W. Alkali Reactivity of Carbonate Rocks-Expansion and Dedolomitization. Highway Research Board Proc. Vol. 40, pp. 462-474, 1961.
5. Haw, V.A. Concrete Aggregate Production. Canadian Mining and Metallurgical Bull., pp. 602-608, Oct. 1957.
6. Hewitt, D.F. The Limestone Industries of Ontario. Ontario Dept. of Mines, Toronto, 1960.
7. Krumbein, W.C. and Pettijohn, F.J. Manual of Sedimentary Petrography. New York, Appleton-Century-Crofts, 1938.
8. Lenart, W.B. Sand and Gravel. Reviews in Eng. Geol. I, pp. 187-196, 1962.
9. Mather, B. Physical Tests on Concrete and Mortar, The Alkali-Aggregate Reaction in Concrete. Highway Research Board Research Report 18-C, p. 21, 1958.
10. Mather, K. Applications of Light Microscopy in Concrete Research, Symposium on Light Microscopy. ASTM Spec. Publ. 132, pp. 51-69, 1953.
11. Mather, K. Crushed Limestone Aggregates for Concrete. Trans. AIME, Mining Eng., pp. 1022-1028, Oct. 1953.
12. Mielenz, R.C. Petrographic Examination of Concrete Aggregates. Geol. Soc. Am., Bull., Vol. 57, pp. 309-318, 1946.
13. Mielenz, R.C. Petrography Applied to Portland Cement Concrete. Reviews in Eng. Geol. I, pp. 1-38, 1962.
14. Mustard, J.N. and Parker, W.E. Location and Evaluation of Concrete Aggregates. Ontario Hydro Research News, Vol. 7, pp. 11-18, 1955.
15. Orchard, D.F. Concrete Technology. New York, Wiley and Sons, Vol. 2, 1962.
16. Powers, M.C. A New Roundness Scale for Sedimentary Particles. Jour. of Sedimentary Petrology, Vol. 23, pp. 117-119, 1953.
17. Swenson, E.G. A Reactive Aggregate Undetected by ASTM Tests. ASTM Bull. 266, pp. 48-51, 1957.
18. Swenson, E.G. and Gillott, J.E. Characteristics of Kingston Carbonate Rock Reaction. Highway Research Board Bull. 275, pp. 18-31, 1961.
19. Glossary of Geology and Related Sciences. Am. Geol. Inst., J.V. Howell, Chairman, NAS-NRC Publ. 501 Washington, D.C., 1957.
20. Book of ASTM Standards, Pt. 4, 1961.
21. Selection and Use of Aggregates for Concrete. ACI Jour., Vol. 58, pp. 513-541, 1961.
22. Department of Highways of Ontario Specifications, Form 310, 1961, Form 1002, 1963.
23. Ontario Hydro Standard Specifications for Road Material, W-3-53, W-10-53, W-11-53, 1953.
24. Symposium on Alkali-Carbonate Rock Reactions. Highway Research Record No. 45, 1964.
25. Comparison Charts for Visual Estimate of Percentage Composition. Washington, D.C., American Geological Institute.

The Skid-Resistant Properties of Carbonate Aggregates

J. E. GRAY, Engineering Director, and
F. A. RENNINGER, Geologist, National Crushed Stone Association

Pavement skid resistance is one of the major problems facing the highway engineer today. During the past fifteen years this problem has generated increased concern, and considerable time and effort have been expended seeking a solution. Unfortunately, during this same period the carbonate aggregates have become almost synonymously associated with pavement slipperiness. The data demonstrate the variability on composition and performance of the aggregates included within this broad trade classification. The skid resistant properties of the carbonate aggregates are shown to be dependent on a differential rate of wear between their constituent minerals and to vary directly with the amount of insoluble sand size material incorporated within the aggregate.

•SKID RESISTANCE has long been recognized as one of the significant properties of any highway surface. The highway engineers of 40 and 50 years ago were concerned with such problems as slippery brick, stone block, and poorly designed asphaltic surfaces. With the introduction of portland cement concrete pavements and better designed bituminous surfaces, this problem was felt to have been eliminated. However, as the volume and speed of the nation's traffic increased, even these pavement types did, in some instances, polish and become excessively slippery (1).

Some 30 years ago Moyer (2) noted that certain aggregates displayed a tendency to polish under the influence of traffic, but it has been only during the past 15 years that any real concern regarding the polishing of highway aggregate materials has been expressed. During this period it has been well substantiated that pavement skid resistance is a function of the aggregate material incorporated within the surface. It was found that some aggregates polished more readily, or to a greater degree, than others. Apparently, therefore, the polishing tendency of a given aggregate is one of its distinct physical properties and as such must be dependent on some characteristic of that material. The most obvious characteristic of an aggregate which might impart such a physical property would be its petrology, or mineral assemblage. Aggregates composed of the harder minerals might be expected to polish at a somewhat slower rate that those composed of the softer minerals. Both, however, might be expected, in time, to produce similar highly polished surfaces. Aggregates composed of minerals possessing varying degrees of hardness could be expected to display differential polishing, that is, the softer materials would polish more rapidly than the harder, leaving a somewhat rough, uneven, and possibly nonskid surface. The British (3, 4) have done some outstanding work regarding the polishing tendencies of highway aggregate materials and their related petrology. Their work has essentially verified the foregoing statements. In 1959, Maclean and Shergold (3) stated the following conclusion: "The petrological characteristics that determine whether a stone will polish or not under pneumatic tyred traffic are not yet known, but it is suggested that one important characteristic of rocks that remain rough is the presence of two minerals that have a considerable difference in their resistances to wear." This work was followed by that of Knill (4), who in 1960 conducted a rather complex investigation of the dependence of an

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Paper sponsored by Committee on Mineral Aggregates.

aggregate's ability to resist polishing on its petrology. She broke the common highway aggregates into several petrologic groups and studied each group separately. The general conclusions of this study were that

> In the igneous rocks, those petrological features which probably affect P (polished stone coefficient) most readily are a variation in the hardness between the minerals, and the proportions of soft minerals present. Rocks in which cracks and fractures are present in the individual mineral grains generally have higher P since such grains are weak and pluck out of the matrix. Finer-grained allotriomorphic igneous rocks, provided they are reasonably fresh, present a tough, cohesive surface which polishes considerably. Sedimentary rocks are variable in their behavior. The Gritstones give a uniformly high P since hard crystals and lithic fragments pluck out of a generally soft and friable matrix. The flints being of simple mineral composition, cryptocrystalline and of uniform hardness polish and have a low P. The Limestone group has an average P of about 0.4. Higher coefficients, in this trade group, appear to be related to the presence of an insoluble residue (especially if such residue is quartz), and to the presence of coarsely crystalline patches of carbonate. The metamorphic rocks tested included members of the Quartzite and Hornfels groups. The intermediate value of P in the Quartzite group is due to the presence of altered feldspars and shattered grains of quartz and quartzite plucking out of a more resistant matrix. However, those quartzites which have recrystallized to form a voidless quartz mosaic can be expected to polish considerably. The polishing of the hornfels is possibly the result of fine grain size and the presence of a high proportion of minerals whose hardness is greater than 5 on Moh's scale, but these observations are based on only three specimens.

The matter of differential hardness seems to be emphasized in the foregoing comments and could conceivably be the controlling factor with regard to aggregate slipperiness.

Carbonate rocks, both limestones and dolomites, form the bulk of the highway aggregates used in this country. Since they are the predominant rock type in most areas, it is economically desirable to make as much use of them as possible. However, during the past 15 years carbonate aggregates have become associated with pavement slipperiness, and their use in the surface course of pavements has been either completely restricted or severely limited in many areas. That many carbonate aggregates do polish and do develop excessively slippery surfaces cannot be denied; however, there are many which have excellent nonskid properties. Limestones and dolomites encompass a wide range of individual rocks and as such might be expected to display many differences with regard to their polishing characteristics. The British investigations indicated that limestones as a group possess low to intermediate or satisfactory resistance to polishing. Investigations during the past several years at the National Crushed Stone Association's laboratory have indicated that aggregate materials bearing the general name "limestone" have developed pavement surfaces varying from excessively slippery to exceedingly nonskid. Corrective measures which will insure the maximum use of even the most polish-susceptible carbonate aggregates have been reported by Gray and Goldbeck (5) in 1959 pertaining to bituminous-concrete pavements, and by Gray and Renninger (6) in 1963 pertaining to portland cement concrete pavements. However, there still exists a need for determining before actual use, or proposed use, the probable skid resistant properties of carbonate aggregates. In 1960 Gray and Renninger (7) presented the results of a preliminary investigation pertaining to the petrological characteristics of carbonate aggregates which apparently are responsible for their behavior with regard to polishing under the action of traffic. This paper is an extension of that initial work and includes the results of that first work as well as that of subsequent investigations.

CARBONATE ROCK DEFINED

Before proceeding further it may be well to establish the meaning of the term limestone, as normally used in the mineral aggregates industry. The term is generally used to identify all carbonate aggregate materials with little, if any, distinction being made between the numerous types of true limestones and dolomites included within this broad category.

Geologically, true limestone is a sedimentary rock, the carbonate portion of which is composed essentially of the mineral calcite ($CaCO_3$) and in which the carbonate fraction exceeds the noncarbonate constituents. The term dolomite should be reserved for those carbonate rocks in which the carbonate fraction is composed primarily of the mineral dolomite ($CaMg[CO_3]_2$). Since most dolomites are postdepositional replacements of limestones, a complete series of rocks can exist between the two extreme compositions. In addition, both limestones and dolomites may contain up to 50 percent of some noncarbonate impurities. Materials such as clay, quartz, chert, chalcedony, pyrite, and feldspar may be included within the makeup of a particular carbonate aggregate. The foregoing definitions suggest the probability of the occurrence of a great number of individual carbonate rock varieties which could vary considerably in their mineral composition and relative Moh's hardness characteristics. Therefore, it is reasonable to expect these carbonates to possess significantly differing polishing characteristics.

DESCRIPTION OF TESTING PROGRAM

Since differential hardness was believed to be the controlling factor with regard to a given carbonate aggregate's susceptibility to the polishing action of rubber-tired traffic, several carbonates possessing noncarbonate fractions of widely differing amounts and of divergent mineralogies were chosen for study. During the past four years, four separate investigations have been conducted with both portland cement and bituminous concretes. Unfortunately, however, during this same period the laboratory apparatus used to measure the skid resistance of the various test surfaces underwent a gradual transition making it impossible to correlate results directly from one group of tests to another. The information obtained, though, was still relative within any given group and the factors controlling skid resistance can be expressed. For this reason, each of the four test series is described and discussed separately, but the results common to all are explained.

Test Series No. 1

The polishing characteristics of five dissimilar carbonate aggregates were studied in bituminous concrete surfaces. The aggregate gradations and mix compositions are given in Table 1. Each mix was designed by the Marshall procedure (ASTM D 1559) and complied with the specifications applicable to the area from which the sample was obtained. Both coarse and fine portions of the aggregates were derived from the same parent rock.

Test Series No. 2

Three carbonate aggregates in the form of manufactured stone sand were studied in portland cement mortar surfaces; the aggregate gradations are given in Table 2 and the mortar batch quantities in Table 3. Each mortar was designed to have the same properties as a mortar which would be worked to the surface during the normal finishing operations of a portland cement concrete pavement. They were designed with sufficient water and entrained air to yield, when combined with the correct amount of 2 in. to No. 4 coarse aggregate, paving concrete having 3 to 6 percent entrained air, and a slump of approximately 2 in.

Test Series No. 3

The polishing characteristics of eight carbonate aggregates in the form of manufactured stone sand were studied in portland cement mortar surfaces designed in a manner

TABLE 1

AGGREGATE GRADATION AND BITUMINOUS MIX COMPOSITION—TEST SERIES NOS. 1 AND 4

Identification	Gradation: Total Percent Passing:													Asphalt Content (%)	Slipperiness (wheel angle deg)
	½ In.	⅜ In.	¼ In.	No. 4	No. 8	No. 10	No. 20	No. 30	No. 40	No. 50	No. 80	No. 100	No. 200		
(a) Test Series No. 1															
1-A	100	99		83		64			21		11		6	6.5	52
1-B	100	90		60		42			18		9		6	5.3	51
1-C	100	90		60		42			18		9		6	5.5	79
1-D	100	95		66		46			18		11		6	6.5	39
1-E	100	95		75		50			17		10		7	6.5	89
(b) Test Series No. 4															
1-B	100	90	-	60	-	42	-	-	18	-	9	-	6	5.3	48
4-A	-	100	-	64	40	-	-	18	-	13	-	10	8	6.0	75
4-B	-	-	100	-	-	94	-	41	-	20	16	-	10	8.0	48
4-C	100	-	73	-	34	-	17	-	-	-	6	-	4	5.0	64
4-D[a]	-	-	-	100	85	-	-	29	-	16	-	10	7	8.0	57

[a] Sheet asphalt mix; aggregate less filler added to make grading here was tested in portland cement mortar.

TABLE 2

AGGREGATE GRADATION AND CERTAIN PHYSICAL PROPERTIES—TEST SERIES NOS. 2 AND 3

Identification	Gradation: Total Percent Passing:							Fineness Modulus	Specific Gravity		Absorption (%)
	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200		Bulk dry	Apparent	
(a) Test Series No. 2											
2-A	-	100	56	28	13	5	3.4	2.98	2.67	2.72	0.76
2-B	100	97	62	39	22	11	4.9	2.70	2.68	2.72	0.61
2-C	100	96	56	31	17	8	5.9	2.92	2.65	2.73	1.11
(b) Test Series No. 3											
3-A	100	83	39	20	11	6	3.1	3.41	2.60	2.77	2.44
3-B	100	98	61	37	21	10	5.0	2.74	2.58	2.72	1.87
3-C	100	91	72	37	15	8	6.1	2.77	2.63	2.72	1.29
3-D	100	99	66	31	16	9	6.7	2.81	2.74	2.82	1.00
3-E	100	92	59	34	18	8	4.0	2.89	2.68	2.72	0.63
2-A	-	100	56	28	13	5	3.4	2.98	2.67	2.72	0.76
2-C	100	96	56	31	17	8	5.9	2.92	2.65	2.73	1.11
1-D	100	75	51	32	19	8	4.8	3.15	2.68	2.74	0.76

TABLE 3

PORTLAND CEMENT MORTAR BATCH QUANTITIES—TEST SERIES NOS. 2 AND 3

Identification	Design Cement Factor (sk/cu yd of conc.)[a]	Actual Batch Proportions (lb/cu yd of conc.)[a]			Actual Cement Factor (sk/cu yd)[a]	Actual Water Content (gal/cu yd)[a,b]	Actual Water-Cement Ratio (gal/sk)[a,b]	Air Content (%)[c]	Slipperiness (wheel angle deg)
		Cement	Water[b]	Carbonate Sand					
(a) Test Series No. 2									
2-A	6.0	560	230	1,090	5.95	27.8	4.67	4.9	48
2-B	6.0	570	235	1,045	6.07	28.3	4.66	3.8	57
2-C	6.0	570	235	1,090	6.09	28.4	4.66	3.8	60
(b) Test Series No. 3									
3-A[d]	6.0	540	223	1,136	5.75	26.8	4.66	6.7	56
3-B	6.0	560	231	994	5.95	27.8	4.67	4.9	66
3-C	6.0	568	235	1,036	6.04	28.2	4.67	4.2	81
3-D	6.0	563	233	1,080	5.99	27.9	4.66	4.6	68
3-E	6.0	566	234	1,084	6.02	28.1	4.67	4.3	61
2-A	6.0	550	227	1,072	5.84	27.3	4.67	5.8	49
2-C	6.0	565	234	1,078	6.02	28.0	4.65	4.4	67
1-D[e]	6.0	569	257	1,164	6.05	30.8	5.09	2.8	40

[a] Based on paving concrete containing 2 in. -No. 4 corase aggregate.
[b] Net water content; does not include water for absorption of aggregates.
[c] Air content expressed as air content of equivalent concrete mix; calculated from measured air content of mortar.
[d] Air content too high; therefore cement factor and water content too low.
[e] Air content too low; water content too high; result of hand mixing.

identical to that previously described. The aggregate gradations are given in Table 2 and the mortar batch proportions in Table 3.

Test Series No. 4

The skid resistant properties of five carbonate aggregates were investigated. Four

TABLE 4

AGGREGATE GRADATION AND PORTLAND CEMENT MORTAR BATCH QUANTITIES

Aggregate Gradation	4-D
Total percent passing:	
No. 4	100
No. 8	81
No. 16	42
No. 30	22
No. 50	12
No. 100	7
No. 200	5.4
Fineness modulus	3.36
Specific gravity	
Bulk dry	2.63
Apparent	2.69
Absorption (%)	0.85
Portland cement mortar batch quantities	
Design cement factor (sk/cu yd of conc.)[a]	6.0
Actual batch proportions (lb/cu yd of conc.)[a]	
Cement	568
Water[b]	235
Carbonate sand	1,196
Actual cement factor (sk/cu yd)[a]	6.04
Actual water content (gal/cu yd)[a, b]	28.2
Actual water-cement ratio (gal/sk)[a,b]	4.67
Air content (%)[c]	4.2
Slipperiness (wheel angle deg)	54

[a] Based on paving concrete containing 2 in.-No. 4 coarse aggregate.

[b] Net water content; does not include water for absorption of aggregates.

[c] Air content expressed as air content of equivalent concrete mix; calculated from measured air content of mortar.

were studied in bituminous-concrete surfaces and the fifth in both a sand-asphalt and a portland cement mortar surface. The portland cement mortar surface was designed as those described previously, whereas the bituminous surfaces were designed in accordance with the specification applicable to the particular areas from which the aggregates were obtained. The aggregate gradations and bituminous mix compositions are given in Table 1, and Table 4 indicates the aggregate characteristics and batch quantities of the one aggregate tested in portland cement mortar.

After placement in the NCSA Circular Test Track (described in the March 1959 issue of the Crushed Stone Journal) the bituminous mixtures were initially rolled with a heavy steel roller and then with a pneumatic tire roller. The portland cement mortar surfaces were hand-finished with a wooden float and allowed to cure under a covering of damp burlap for a period of 28 days. Following this initial treatment both the bituminous and portland cement surfaces were subjected to thousands of passes of a wheel equipped with a rubber tire. Initially, water and a fine siliceous sand were placed on the test surfaces as an aid to the tire in wearing away the surface binder (either asphalt or portland cement paste) and exposing the aggregates. Once the aggregates were exposed the surfaces were cleaned and dried. To polish the exposed aggregate particles the traffic was continued using only the rubber tire.

DETERMINATION OF SLIPPERINESS

At appropriate intervals throughout the duration of each test series measurements of the skid resistance of the various test sections were made with the NCSA Slipperiness Testing Apparatus. A description of the present device and the operating procedure employed are given in the Appendix. Unfortunately, as mentioned previously, the Slipperiness Testing Apparatus underwent a period of transition during the tenure of the tests. The present apparatus was used to determine the skid resistance of those test sections comprising Test Series Nos. 2, 3, and 4. The same device, somewhat modified, was used during the conduct of Test Series No. 1. In place of the rubber strip on the present tire a portion of the original tread was left intact, the weight assembly (driving force) was somewhat heavier, and the tire rim was of a lighter design. These differences in design result in a wider spread, and generally higher values of wheel angle readings for the Test Series No. 1 surfaces. Both wheels have been found to correlate well with large-scale stopping-distance vehicles at various times; however, their measurements, due to tire wear, should be interpreted only as relative within a given series of tests. For these reasons no attempt has been made to correlate results from one test series to another. A continuous calibration of the device, during any one test series, was maintained by periodic tests on ground glass plate "standards of slipperiness."

This work was completed before the time the British Portable Skid Tester was commercially available. The British Tester which is presently employed in the NCSA laboratory has been found to correlate fairly well with the Bicycle Wheel Apparatus during any specific test series.

RATE OF POLISH

Certain carbonate aggregates polish quite rapidly with the application of very little traffic whereas others do not polish or polish very slowly during even extended periods of traffic application. This statement is amply supported by Figures 1 through 4 which depict the polishing rates of the various carbonate aggregates studied under each of the four test series. Several years ago questions were raised concerning the criteria or method needed to determine when this polishing traffic should be stopped. Ordinarily tests of this nature had been discontinued when the polishing rate curve became essentially horizontal. This normally occurred between 40,000 and 60,000 passes of the rubber tire. The assumption in the past was that at such a point a more or less static condition of slipperiness had been reached; however, this assumption had never been verified. To justify the stopping of the tests at such a point the polishing traffic applied to the surfaces constituting Test Series No. 1 was purposely continued well beyond the leveling-off point. Figure 5 shows that very little additional polishing was achieved once the leveling occurred at about 40,000 passes of the wheel. The use of the wheel angle readings obtained at the leveling-off point, 40,000 passes in this case, as a measure of the ultimate skid resistance of the various test surfaces was enhanced by the fact that these values all lay within one standard deviation of the values obtained after extending the traffic an additional 55,000 passes. The dotted bands in Figure 5 are the standard deviation limits of the readings made after 95,000 passes of the wheel for each of the test sections shown.

METHOD OF STUDYING CARBONATE ROCK CHARACTERISTICS

Since the polishing characteristics and the ultimate level of skid resistance attained by carbonate aggregates were believed to be dependent on some definite physical property or properties of the aggregate in question, a systematic procedure had to be developed to determine the basic properties of these materials. The individual samples studied were representative of stockpiles of commercially produced stone including material from the entire working face of the various quarry operations. After some exploratory tests, the following procedure was adopted:

1. Samples of each aggregate, representative of their respective gradations as used, in either the bituminous concretes or portland cement mortars, during the skid resistance studies, were selected.

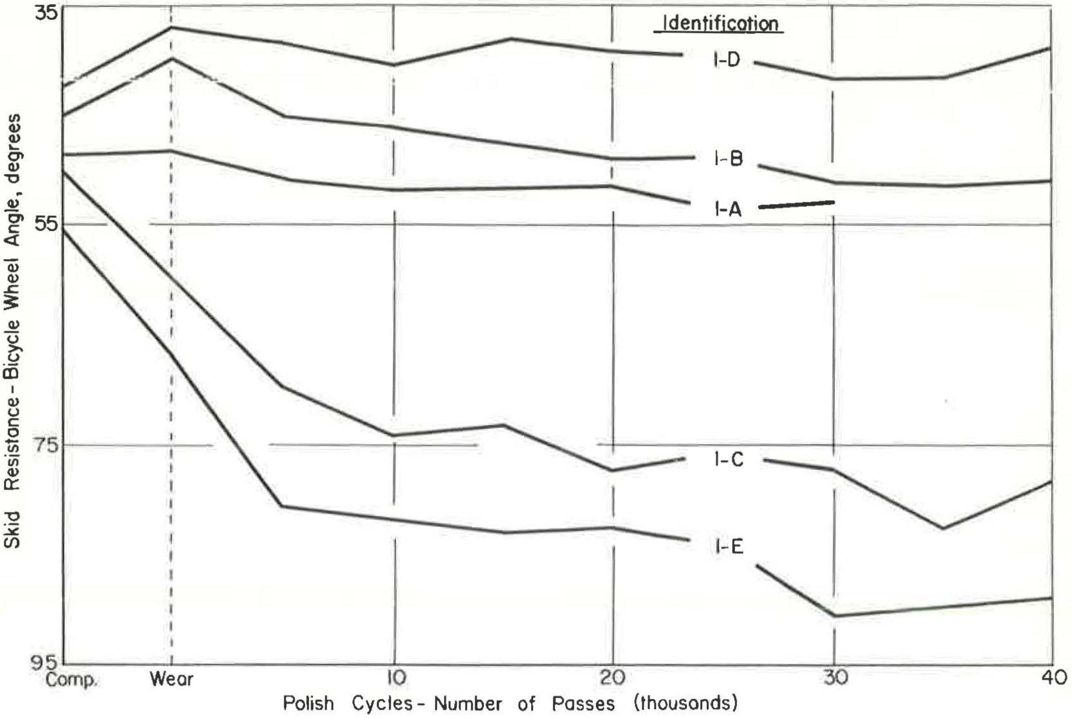

Figure 1. Rate of polish, Test Series No. 1.

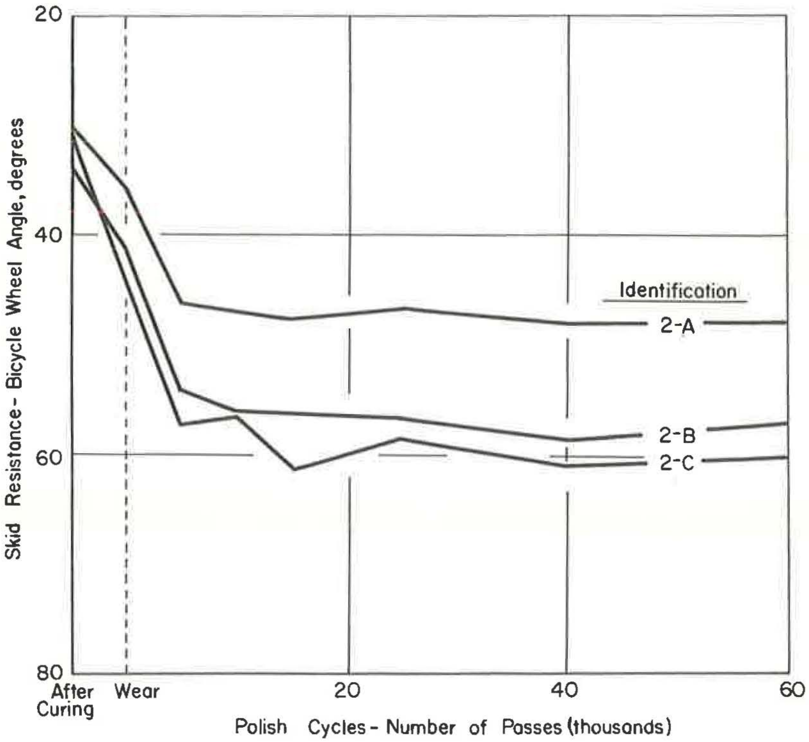

Figure 2. Rate of polish, Test Series No. 2.

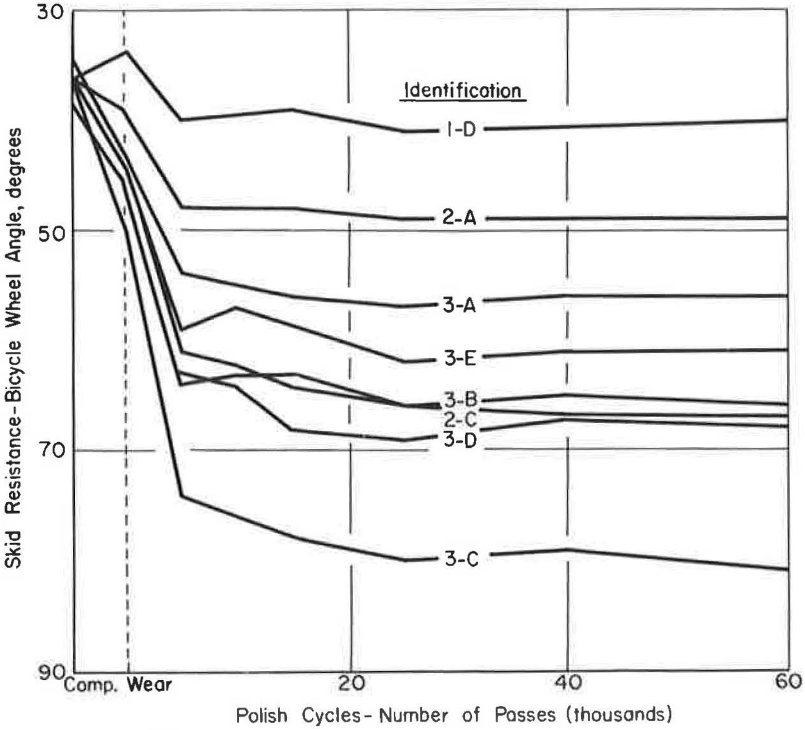

Figure 3. Rate of polish, Test Series No. 3.

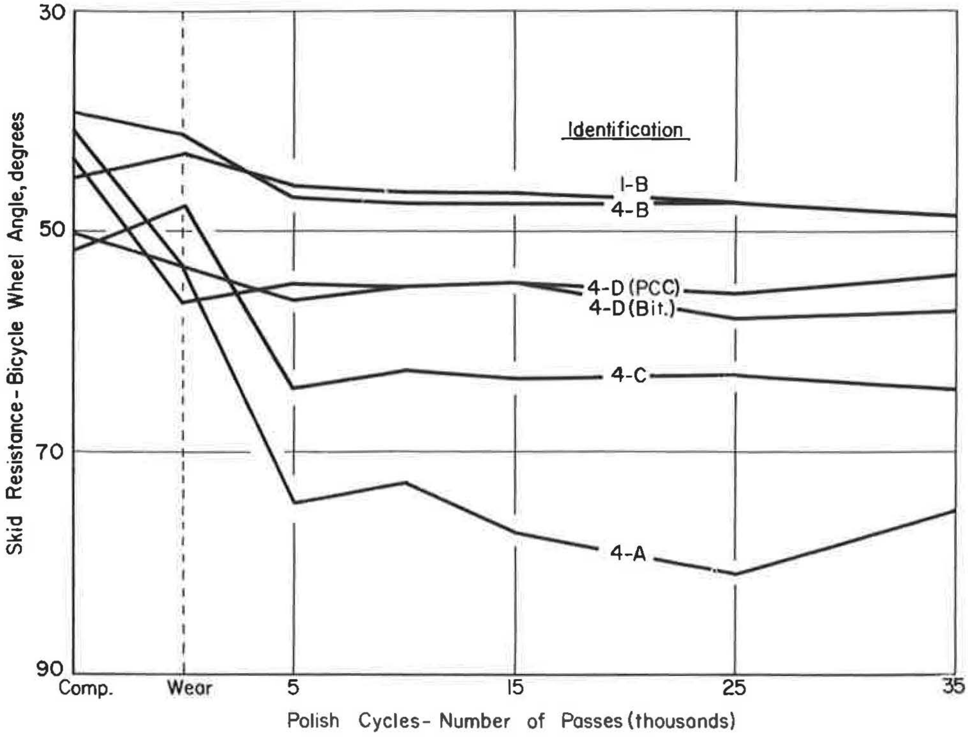

Figure 4. Rate of polish, Test Series No. 4.

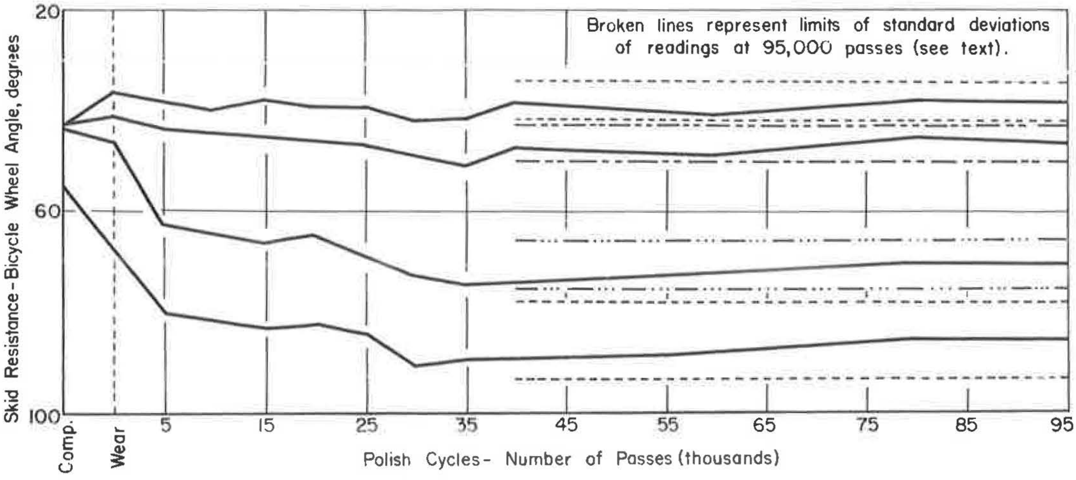

Figure 5. Effect of traffic on ultimate skid resistance.

2. Each of the samples was reacted with a dilute solution of hydrochloric acid until the carbonate minerals had been fully digested.
3. The residue remaining after the acid leaching was filtered, washed, dried, and weighed, and the percentages of total carbonates and insoluble residue calculated.
4. The insoluble residue was then tested for grain-size distribution using ASTM D 422, Grain Size Analysis of Soils (Tentative), and for specific gravity by ASTM D 854, Specific Gravity of Soils.
5. An approximate determination of the mineral composition of each insoluble residue was made under a low power binocular microscope.

The results of the insoluble residue determinations and the grain-size analyses of those residues, Tables 5 and 6 indicate, to some extent, the heterogeneous nature of carbonate rocks as a group. A classification of the insoluble residues as percentages of the total aggregate, using a soils classification of sand size, silt size, and clay size, is also given. The insoluble residue coarser than the silt fraction, that is, greater than 0.05mm, was studied microscopically in an effort to describe the accessory mineral composition of each carbonate aggregate under study (Tables 5 and 6).

INFLUENCE OF CARBONATE ROCK CHARACTERISTICS ON SLIPPERINESS

The results obtained for the group of aggregates in Test Series No. 1 (the initial investigation previously reported by Gray and Renninger in 1960) are discussed in some detail and the results of the remaining three test series are used to substantiate the original conclusions. However, despite the fact that skid resistance is expressed as a bicycle wheel angle in each case, the interseries results should not be directly correlated with one another on that basis. As explained earlier, the NCSA Slipperiness Testing Apparatus underwent some change during the conduct of these investigations and the results are relative only within any one given series of tests.

Test Series No. 1

Carbonate aggregate 1-C which proved to be quite slippery was a siliceous carbonate containing approximately 25 percent acid insoluble residue, 40 percent of which (i. e., 10 percent of the total rock) was composed of extremely fine material, 0.05 mm or less in diameter. Carbonate aggregate 1-E, the most slippery of the five aggregates in this test series, possessed the smallest percentage of acid insoluble residue. Two-thirds of its 9 percent residue was composed of material of the silt and clay size range, i. e, less than 0.05 mm in diameter. Therefore, of the two aggregates which proved to be the most slippery in this series of tests, one had a low residue percentage, whereas the

TABLE 5

GRADATION AND SIZE CLASSIFICATION OF INSOLUBLE RESIDUE

Identification	Slipperiness (wheel angle deg)	Carbonate (%)	Acid (%)	Grain Size Tot. Percent Finer Than:										Size Classification (% of Total Aggr.)				Total
				3/8 In. (9.52 mm)	No. 4 (4.76 mm)	No. 10 (2.00 mm)	No. 40 (0.42 mm)	No. 60 (0.25 mm)	No. 80 (0.177 mm)	No. 100 (0.149 mm)	No. 200 (0.074 mm)	(0.05 mm)	(0.005 mm)	Coarse Aggr. (+2.0 mm)	Sand (0.05 to 2.0 mm)	Silt (0.005 to 0.05 mm)	Clay (-0.005 mm)	
									(a) Test Series No. 1									
1-A	52	58.5	41.5	100	90	78	47	39	33	-	21	14	2	9.3	26.1	5.1	1.0	41.5
1-B	51	64.2	35.8	100	94	85	63	51	42	-	25	14	2	5.4	25.2	4.3	0.9	35.8
1-C	79	74.6	25.4	100	81	69	52	48	47	-	41	40	16	7.9	7.3	6.3	3.9	25.4
1-D	39	51.9	48.1	100	100	98	94	78	64	-	18	15	8	0.9	40.1	3.5	3.6	48.1
1-E	89	90.8	9.2	100	100	94	85	81	78	-	66	60	20	0.6	3.1	3.7	1.8	9.2
									(b) Test Series No. 2									
2-A	48	85.2	14.8	-	100	100	97	77	62	-	40	34	7	0.0	9.8	4.0	1.0	14.8
2-B	57	85.0	15.0		100	97	82	76	72	-	62	55	5	0.4	6.4	7.5	0.7	15.0
2-C	60	83.7	16.3		100	100	94	91	90	-	84	79	23	0.0	3.5	9.1	3.7	16.3
									(c) Test Series No. 3									
3-A	56	94.4	5.6		100	84	60	-	50	-	44	42	5	0.9	2.3	2.1	0.3	5.6
3-B	66	97.5	2.5		100	73	56	-	45	-	30	30	14	0.7	1.1	0.4	0.3	2.5
3-C	81	95.7	4.3		100	100	96	-	91	-	84	81	50	0.0	0.8	1.3	2.2	4.3
3-D	68	96.0	4.0		100	100	95	-	90	-	84	77	14	0.0	0.9	2.5	0.6	4.0
3-E	61	84.2	15.8		100	99	83	-	71	-	59	52	7	0.2	7.3	7.2	1.1	15.8
2-A	49	85.2	14.8		100	100	97	-	62	-	40	34	7	0.0	9.8	4.0	1.0	14.8
2-C	67	83.7	16.3		100	100	94	-	90	-	84	79	23	0.0	3.5	9.1	3.7	16.3
1-D	40	51.9	48.1		100	98	94	-	64	-	18	15	8	1.0	39.9	3.4	3.8	48.1
									(d) Test Series No. 4									
4-A	75	86.2	13.8	100	100	83	67	-	59	57	51	39	-	2.4	6.0	5.4[b]		13.8
4-B	48	57.4	42.6	100	100	93	45	-	29	25	16	10	-	3.0	35.1	4.5[b]		42.6
4-C	64	92.9	7.1	100	98	91	82	-	78	76	63	46	-	0.6	3.3	3.2[b]		7.1
4-D	57	64.3	35.7	100	100	77	25	-	13	11	7	6	-	8.1	25.6	2.0[b]		35.7
4-D[a]	54	63.9	36.1	100	100	77	23	-	12	10	7	6	-	8.4	25.8	1.9[b]		36.1
1-B	48	64.2	35.8	100	94	85	63	-	42	-	25	14	-	5.4	25.2	5.1[b]		35.8

[a] Used in portland cement mortar surface. [b] Silt and clay, -0.05 mm.

TABLE 6

IDENTIFICATION OF AGGREGATE BASED ON MINERAL COMPOSITION

Identification	Approximate Mineral Composition	Percent	Name
	(a) Test Series No. 1		
1-A	Carbonates Quartz Mica minerals Feldspar Clay or clay-like material Carbonaceous material	58 17 11 10 4	Arenaceous carbonate
1-B	Carbonates Quartz Mica minerals Magnetite	64 10 26 Trace	Mica rich siliceous carbonate
1-C	Carbonates Quartz Mica minerals Clay or clay-like material	75 25	Grey siliceous carbonate
1-D	Carbonates Quartz Mica minerals Carbonaceous material	52 47 1	Arenaceous carbonate
1-E	Carbonates Insoluble residue (Too fine grained to distinguish under 30x)	91 9	Grey carbonate
	(b) Test Series No. 2		
2-A	Carbonates Acid insoluble residue Primarily discrete particles of quartz resembling a fine beach sand, with minor amounts of clay or clay-like material and a trace amount of biotite mica.	85. 2 14. 8	Siliceous carbonate sand
2-B	Carbonates Acid insoluble residue Appreciable amounts of quartz, clay or clay-like material, with minor constituents including chert, pyrite, magnetite and some limonite.	85. 0 15. 0	Siliceous carbonate sand
2-C	Carbonates Acid insoluble residue Very small quartz particles, 30 to 40 percent clay or clay-like material, with trace amounts of mica.	83. 7 16. 3	Siliceous carbonate sand
	(c) Test Series No. 3		
3-A	Carbonates Insoluble residue Contains a considerable amount of siliceous fossil replacements and clay or clay-like material, with discrete particles of both quartz and magnetite also evident; many particles sub- to well-rounded in shape.	94. 4 5. 6	Carbonate sand
3-B	Carbonates Insoluble residue Primarily sharp, angular chert particles resembling quartzite, and discrete quartz particles, with trace amounts of pyrite.	97. 5 2. 5	Carbonate sand

Identification	Approximate Mineral Composition	Percent	Nam
	(c) Test Series No. 3 (cont'd.)		
3-C	Carbonates Insoluble residue Almost entirely finely divided quartz, and clay or clay-like material, medium to dark grey, with trace amounts of an iron-bearing mineral, probably limonite, and minor amounts of both biotite and muscovite mica.	95. 7 4. 3	Carbonate sa
3-D	Carbonates Insoluble residue Primarily clay or clay-like material, with minor amount of discernible quartz particles showing an iron stain; trace amounts of pyrite, mica, and an iron mineral, probably limonite.	96. 0 4. 0	Carbonate sa
3-E	Carbonates Insoluble residue Appreciable amounts of quartz and clay or clay-like material, with minor constituents including chert, pyrite, magnetite, and probably limonite.	84. 2 15. 8	Siliceous Carbonate
2-A	See 2-A, Test Series No. 2		
2-C	See 2-C, Test Series No. 2		
1-D	Carbonates Insoluble residue Almost entirely discrete quartz particles with trace amounts of both biotite and muscovite mica; resembles a fine beach sand.	51. 9 48. 1	Arenaceous carbonate
	(d) Test Series No. 4		
4-A	Carbonates Insoluble residue Light grey, fine-grained quartz particles and a clay-like material with trace amounts of iron (limonite).	86. 2 13. 8	Siliceous carbonate
4-B	Carbonates Insoluble residue Light to dark grey quartz particles and what appears to be detritus from an acidic metamorphic rock containing feldspar, quartz, mica, and some dark-colored iron minerals.	57. 4 42. 6	Arenaceous carbonate
4-C	Carbonates Insoluble residue Dark grey, fine-grained quartz and a clay-like material, with some limonite and probably dark-colored carbonaceous material.	92. 9 7. 1	Carbonate
4-D	Carbonates Insoluble residue Light to medium grey chert and quartz with some minor amounts of clay, chert, occurring in coarser fractions and quartz in the finer sizes.	63.9-64.3 35.7-36.1	Cherty carbonate
1-B	See 1-B, Test Series No. 2		

second contained an appreciable amount of residue. Therefore, the residue content alone does not control the skid resistance of a carbonate aggregate. However, both of the foregoing aggregates had one factor in common. The grain-size distribution of the residues of these two aggregates (Table 5) indicates that appreciable amounts of both residues are less than 0.05 mm in diameter, i. e., both are composed essentially of silt and clay size material. These same data also serve to indicate that this very fine siliceous material does not effectively contribute to aggregate skid resistance.

Aggregates 1-A and 1-D were arenaceous carbonates and both developed surfaces possessing good skid resistance. Aggregate 1-A contained 42 percent acid insoluble

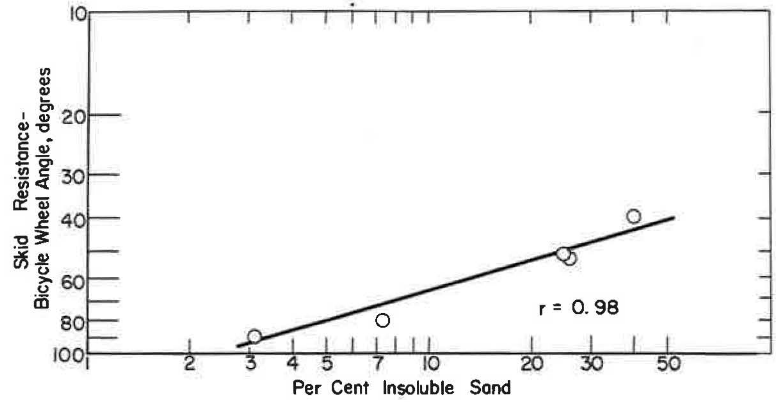

Figure 6. Relationship between skid resistance and amount of insoluble sand, Test Series No. 1.

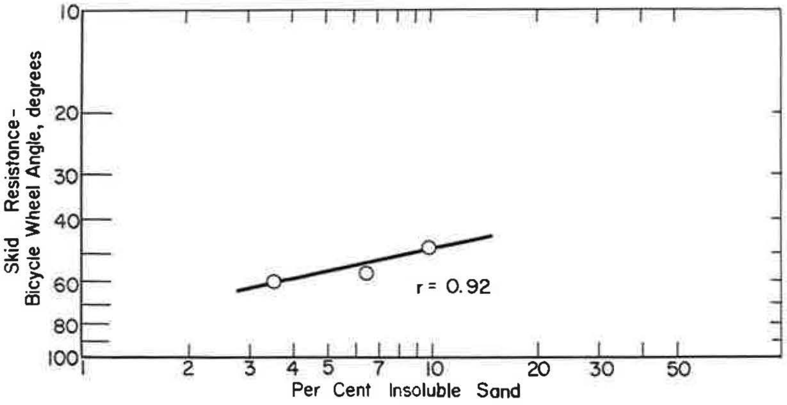

Figure 7. Relationship between skid resistance and amount of insoluble sand, Test Series No. 2.

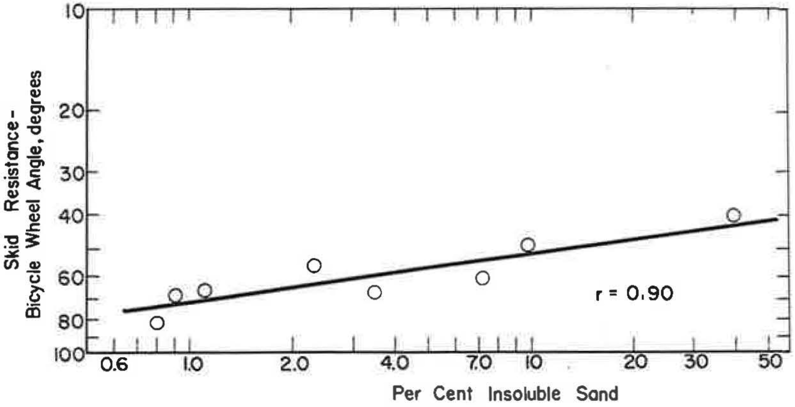

Figure 8. Relationship between skid resistance and amount of insoluble sand, Test Series No. 3.

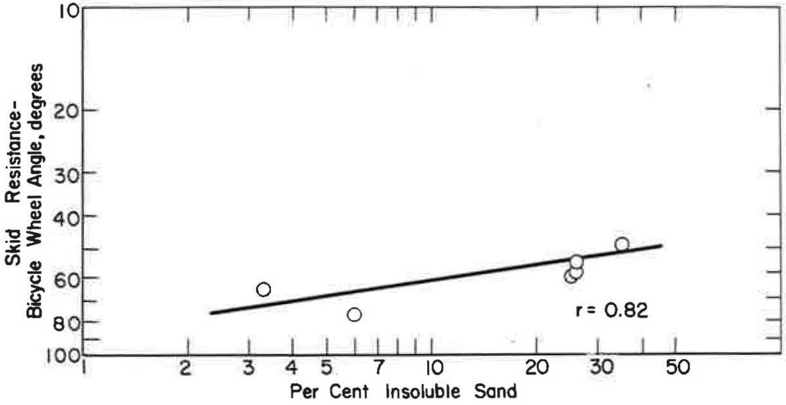

Figure 9. Relationship between skid resistance and amount of insoluble sand, Test Series, No. 4.

residue which was composed primarily of sand size (0.05 to 2.0 mm) grains of quartz, mica, and feldspar. The residue content of aggregate 1-D was 48 percent and was composed almost entirely of sand size grains of quartz. From these results, it would appear that the amount of acid insoluble residue does influence, but not control, the polishing characteristics of carbonate aggregates. There is evidence that the sand-size grains of the residue effectively contribute to the ultimate skid resistance of a carbonate aggregate. It might be expected that a carbonate aggregate having a residue content of 48 percent, most of which was sand size particles of quartz, might have the appearance of a coarse sandpaper, providing the carbonate fraction was sufficiently soft and capable of being worn away to expose the quartz particles. Actually, aggregate 1-D had just such an appearance after being subjected to the rubber tired traffic.

Aggregate 1-B, which also possessed good nonskid properties, was noticeably different from either sample 1-A or 1-D. The acid insoluble residue content of this aggregate was 36 percent but consisted primarily of the mica minerals with some quartz. The grain-size distribution of the residue was, however, quite similar to that of aggregate 1-A and the skid resistance of the bituminous-concrete surfaces containing these two aggregates was very nearly the same despite the fact that the residue of aggregate 1-B was composed essentially of the mical minerals, a rather unusual occurrence for the average carbonate rock, which is one-half to one point softer than carbonate material on the Moh scale of hardness. It would seem, therefore, that the polishing characteristics of carbonate aggregates are not indicated solely by the hardness of the minerals comprising the aggregate, but rather by a differential in the hardness of the mineral constituents. The foregoing statement made as a result of these studies finds support through the works of other investigators. The work of Maclean and Shergold (3) and that of Knill (4) have resulted in similar findings.

It is, however, questionable whether differential hardness in itself is the essential property since the results of this initial study also tend to indicate that it is a combination of differential hardness, amount of acid insoluble residue, and the grain-size distribution of that residue which seems to govern the polishing characteristics of carbonate aggregates. Certainly the presence of even appreciable quantities of hard minerals in the silt and clay size range does not impart nonskid properties to carbonate rock as evidenced by the results obtained with aggregate 1-C in which appreciable quartz is present in these sizes.

Since the results of this initial investigation seemed to define the physical properties necessary to a nonskid carbonate aggregate, and since the plot of bicycle wheel angle or degree of slipperiness vs the percent insoluble sand size material (0.05 to 2.0 mm), Figure 6, showed such a high degree of correlation it was felt some supporting data should be developed. Test series Nos. 2, 3, and 4 were conducted with this in mind.

Test Series No. 2

The three aggregates studied during this portion of the investigation were in the form of manufactured stone sands and were incorporated in portland cement mortars, the design properties of which were described earlier and are given in Table 3. The three carbonate sands contained approximately the same amounts of acid insoluble residue; however, the grain-size distribution of these residues was noticeably different (Table 6). The skid resistance of surfaces containing these particular materials varied inversely with their residue contents, further evidence that residue content alone is not the controlling factor which determines a given carbonate's nonskid properties.

The acid insoluble residue of each of the three aggregates was composed essentially of siliceous materials (Table 6), which differed primarily in their respective grain-size distributions. Figure 7 plots bicycle wheel angle or slipperiness vs the percent of insoluble sand size material. The measured skid resistance can again be seen to vary directly with this particular aggregate characteristic.

Test Series No. 3

This group of tests was essentially an extension of Test Series No. 2 and two of the aggregates included were identical to two (aggregates 2-A and 2-C) studied in that series. Aggregate 1-D, in the form of a stone sand, was also included for comparative purposes. The acid insoluble residues of the eight test aggregates again displayed a considerable variation in quantity and did not, as such, correlate with the skid resistance of the test surfaces. The grain-size distribution of the eight residues varied noticeably from fine to coarse and it was again possible to correlate the insoluble sand content of each aggregate with the skid resistance displayed by its respective test surface. Table 5 contains the pertinent information relative to the insoluble residues of the eight carbonate sands investigated and Figure 8 plots insoluble sand content vs skid resistance.

Test Series No. 4

The four carbonate aggregates (1-B, 4-A, 4-B, and 4-C) displayed the now typical variations in size distribution and amount of acid insoluble residue and skid resistance (Table 6). One aggregate, carbonate aggregate 4-D, was studied in both a sheet asphalt and a portland cement mortar surface. No appreciable difference in the level of skid resistance attained by these two surfaces was recorded (Tables 1 and 4), which may be taken to indicate that the type of binding material has little, if any effect on the ultimate skid resistance developed by a highway surface and that the aggregate comprising that surface is, in fact, the primary contributor to pavement skid resistance or pavement slipperiness. Figure 9 plots the insoluble sand content of each aggregate and the measured skid resistance of their respective test sections made without regard to binder type. The degree of correlation of the data of this test series was somewhat lower than that of the previous three. The calculated correlation coefficient for Test Series No. 4 was 0.82 as compared with 0.98, 0.92, and 0.90 for Test Series No. 1, No. 2, and No. 3, respectively.

GENERAL CONCLUSIONS

Carbonate aggregates are a rather heterogeneous group of materials and, as such, possess varied physical, mineralogical, and textural properties. These differences in turn control the various performance characteristics of the respective aggregate. Not all carbonate aggregates are expected to perform similarly in the many aggregate evaluation tests, nor are they expected to perform the same in portland cement concretes, bituminous concretes, and graded aggregate base courses. The variations present in the many types of carbonate rock determine their suitability for various uses. The rate at which carbonate aggregates polish under the application of rubber tired traffic and the ultimate level of pavement skid resistance they are capable of maintaining is a performance characteristic which varies with the particular aggregate in question and is therefore related in some way to the basic properties of that aggregate.

The data presented here have successfully related certain physical and mineralogical characteristics of carbonate rocks to their ultimate level of pavement skid resistance. The polishing rate and the ultimate skid resistance level of a given carbonate aggregate have been found to vary directly with the percentage of acid insoluble sand size material present in that aggregate. This dependence of skid resistance on insoluble sand content was displayed in each of the four test series discussed. Unfortunately, however, because of the limitations of our present laboratory methods of evaluating skid resistance, it was not possible to make an acceptable correlation from one test series to another.

This work has been confined to the skid resistant properties of impure limestones, i. e., those not of a chemical grade. Shupe and Lounsbury (8) have shown, however, that when dealing with relatively pure carbonate aggregates the skid resistance of bituminous mixtures containing those aggregates was dependent on the calcium carbonate or calcite content. That is, as the calcite content increases, the susceptibility to polishing also increases. No attempt was made during this series of tests to make such an evaluation, nor was the degree of crystallinity considered. The effect these properties might have on the polishing characteristics of impure limestones and dolomites was not established. Indications are, however, that as the dolomite content increases, so does the aggregate's level of skid resistance, at least when considering low residue carbonates. As the residue content increases, its size distribution evidently overshadows these other characteristics, and is the primary skid resistance controlling property.

These data are presented with the hope that specifying agencies will consider some of the many basic differences which result in the varying performance characteristics of carbonate aggregates. Blanket rejections of limestones for skid resistance purposes solely because they belong to this heterogeneous trade name group may result, in many cases, in the refusal to consider perfectly adequate materials.

REFERENCES

1. Dillard, J. H., and Alwood, R. L. Providing Skid-Resistant Roads in Virginia. AAPT Proc., Vol. 26, 1957.
2. Moyer, R. A. Skidding Characteristics of Automobile Tires on Roadway Surfaces and Their Relation to Highway Safety. Bull. 120, Iowa Eng. Exp. Sta., Ames, Iowa, 1934.
3. Maclean, D. J., and Shergold, F. A. The Polishing of Roadstones in Relation to Their Selection for Use in Road Surfacing. Jour., Inst. of Highway Eng., 1959.
4. Knill, D. C. Petrographical Aspects of the Polishing of Natural Roadstones. Journal of Applied Chemistry, 1960.
5. Gray, J. E., and Goldbeck, A. T. Skid Proofing of Asphaltic Concrete Pavement Surfaces. Crushed Stone Jour., March 1959.
6. Gray, J. E., and Renninger, F. A. Limestone and Dolomite Sands in Skid Resistant Portland Cement Mortar Surfaces. Res. Rept. National Crushed Stone Association, Washington, D. C., Sept. 1963.
7. Gray, J. E., and Renninger, F. A. Limestones With Excellent Non-Skid Properties. Crushed Stone Jour., Dec. 1960.
8. Shupe, J. W., and Lounsbury, R. W. Polishing Characteristics of Mineral Aggregates. Presented at First International Skid Prevention Conf., Charlottesville, Va., Sept. 8-12, 1958.

Appendix

DESCRIPTION OF PRESENT NCSA SLIPPERINESS TESTING APPARATUS

The essential parts of the NCSA Slipperiness Testing Apparatus (Fig. 10) consist of a flexible outer frame which supports a rigid inner frame by means of a loose fitting adjusting screw. The axle of the bicycle wheel rests in slotted supports which are a part of the inner frame and permit free vertical motion of the axle. The wheel is a heavy duty bicycle front wheel equipped with a 26 × 2.125 tire with its normal tread removed by a belt sander. A strip of rubber 1 in. wide by $\frac{1}{4}$ in. thick is cemented over half the circumference of the tire. The leading edge of this rubber strip which, during operation of the wheel, first comes in contact with the road surface, is tapered from full thickness to zero thickness over a distance of approximately 5 deg. Tire inflation pressure is 22 psi.

Motive force for the operation of the wheel is provided by a weight consisting of two sections of $\frac{3}{4}$-in. i. d. copper tubing about 8 in. long filled with lead and bent to an outside radius of 10 $\frac{3}{4}$ in. to fit the shape of the tire rim. The weight assembly is secured by two small bolts that extend through the two sections and press them tightly against the spoke nipples. The entire weight assembly weighs 1, 663 g and is centered approximately 50 deg from the leading edge of the rubber strip.

In operation, initial adjustment is made so that when the rubber strip and the weight assembly are in a downward position the rubber strip is barely in contact with the pavement but the wheel is still supported by the axle. The wheel is then rotated 180 deg so that the rubber strip and the weight assembly are in their uppermost position. The wheel is locked in this position and it and the inner frame assembly are lowered $\frac{1}{12}$ in. by means of the adjusting screw. The wheel is then unlocked and rotates freely until

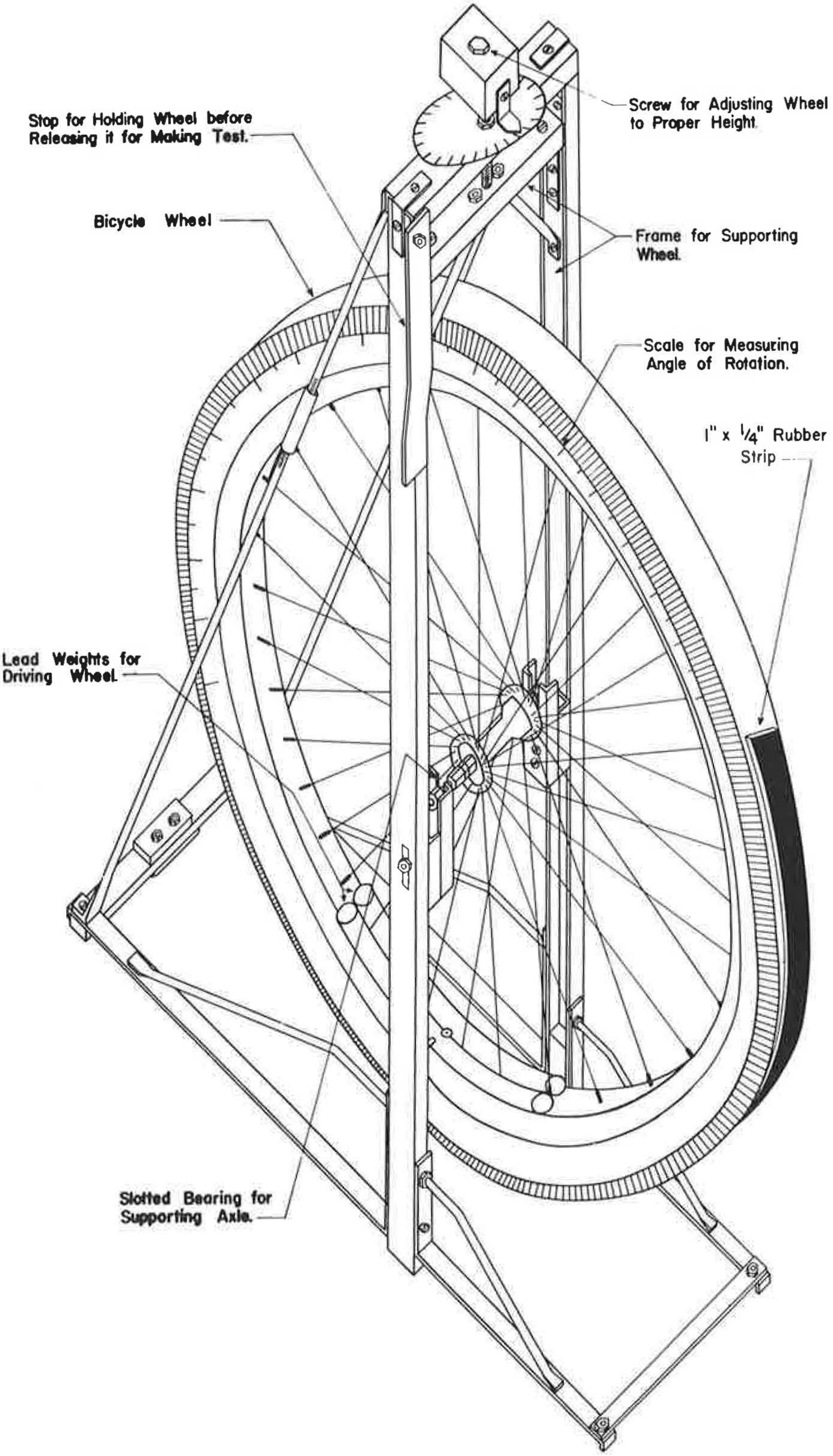

Figure 10. Slipperiness apparatus.

the rubber strip attached to the tire comes in contact with the pavement surface, thus raising the axle off its supports and permitting the tire to slide on the pavement surface until it is brought to rest by the forces of friction.

For any repetition of tests, the angular momentum of the wheel at the instant of contact between tire and pavement surface is the same and the energy expended is reflected in the angular distance through which the wheel turns after striking the pavement surface. Thus, the angle through which the wheel rotates while the tire is in contact with a surface serves as a measure of its skid resistance. The wheel angle readings increase as the skid resistance of the surface decreases. Usually an average of 24 readings in different positions on the road surface is obtained.